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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

This Society is not responsible for any statement made or opinion expressed in its publications.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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BUILDERS, DEFENDERS, AND POLITICAL DESPOILERS OF OUR COUNTRY

ADDRESS AT THE ANNUAL CONVENTION, IN BUFFALO, N. Y., JULY 18, 1928

By Lincoln Bush,* President, Am. Soc. C. E.

The progress of humanity has always depended on two things: First, on the strength of the natural instinct; and, second, on those necessities which a Mother Nature has sternly although kindly imposed upon her children. These two conditions have been the great drive wheels of the human engine, while the human mind—the reasoning power—has been the potent lever which for good or for ill, depending on the skill, the courage, the power, and the inspirations of the engineer, has commanded and controlled the whole machine.

Builders of Our Country

The laws of progress are but a compilation of the unwritten statutes of the Creator. Clearly discernible among these may be noticed the motives, the inspirations, the incentives, and the impelling forces, prompting through all life and ages to beneficence and accomplishment. This is exemplified by our former member, James B. Eads, in the dedication of soul and self to the construction of the Mississippi River jetties in these words,

"I therefore undertake the work with a faith based upon the ever-constant Ordinances of God Himself and so certainly as He spare my life and faculties for two years more, I will give to the Mississippi through His Grace and by the application of His Laws, a deep, open, safe, and permanent outlet to the Seas."

Forty-nine years have since passed and there now remains a clear channel of 40 ft. in depth, at the South Pass Jetties, and Eads' memory is enshrined in the Hall of Fame.

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* Cons. Engr., East Orange, N. J.

The learned explorations, the scientific discoveries, and the mental developments of each succeeding generation are but the outgrowth of a clearer and a fuller understanding of these Laws. What a beneficent creation! With delight we pursue; with gratification, possess. There are no limitations of the materials with which to work; no end of new truths, new laws, and new relations, which intelligent thought and energy cannot develop and grasp. Have you not heard of the new one-span structure reaching from Rocky Point to Rugby, 3 200 miles across the Atlantic? Invisible and intangible, its chords and web members were assembled on the evening and the morning of the Second Day, fifty-nine centuries ago; and engineering science, invention, and research are only now sending the first message across this vibrant mystery. Diamonds at our feet and world without end!

Undaunted by the failure of France, after thirty years of effort and the expenditure of $200 000 000 to build the Panama Canal, our engineers have said to the United States Government, "This is a practicable and a feasible proposition; we can build this great waterway". And within a decade we gave to the world that of which it had dreamed for 400 years, making New York from the Philippines only 9 600 miles instead of 13 600 miles, and the Golden Gate from Liverpool only 8 000 miles instead of 13 800 miles, and we now challenge the world for the billions of commerce of the East and West and for the inter-coastal trade of the two Americas. Conquests of the Civil and Sanitary Engineers: Wallace, Stevens, Goethals, and Gorgas!

The following record of our War Department was made only a few years before the last spike was driven in our first great transcontinental railroad in 1869; a record made by men who believed they were correctly forecasting the future of our present Great West beyond the Missouri and Mississippi Rivers, which reads as follows:

"From these vast prairies will be derived one of the great advantages to the United States, namely, the restriction of her population to some central limits and thereby a continuation of the Union. They [meaning the people] will be constrained to limit themselves to the borders of the Missouri and Mississippi, while they leave the prairies beyond, incapable of cultivation, to the wandering and uncivilized Aborigines of our Country".

Across these arid plains and tables of our Great West, we see in the sands and sage-brush the stakes and footprints of the Civil Engineer; and following in his wake, we find the storage reservoirs, the irrigating channels, the shimmering fields of golden grain, and the Great American Desert is no more.

Instead of these vast prairies, declared incapable of cultivation, we have on this same territory to-day, the largest flouring mills, the greatest stock ranges, the largest granaries, and many of the richest ore mines of the world. Instead of these wandering and uncivilized aborigines we have to the west of the Mississippi River, the homes and firesides of 36 000 000 of our most intelligent people, the backbone of your own and New England's best stock, and heirs of the blood and courage of the pioneer. Instead of the aborigines' tepee, we have to the west of this great river, 284 universities and colleges, a wealth represented by $102 000 000 000 of tangible property, and the emigrants' trail

of '49 has been buried under 127 000 miles of the most modern and well equipped railroad of our Country.

What a marvelous development in our great transportation systems, with only 23 miles of railroad in the United States in 1830 and 250 000 miles of railroad in the United States in 1925! They have stretched out over mountain and plain, up and over the great Rocky Ranges of our West and Northwest into that land of sunshine and flowers, and still on to the Yukon and Circle City, into a country with a coast line of 8 000 miles, of which the half has never been told, with 1 mile of rail for every 470 of our people against 1 mile of railroad for every 2 050 of Continental Europe, without which our great West, beyond the Father of Waters, never could have been and the Louisiana Purchase would have been of little comparative value—a vast empire of 875 000 sq. miles, stretching west from the Mississippi to the Great Continental Divide and north from the Gulf to the Canadian Border. This acquisition barred forever the erection of a hostile or foreign State, made inevitable the annexation of Texas, New Mexico, California, and Oregon, and gave birth to a new nation of continental proportions and world influence.

Some one will say, "capital and labor have done these great and wonderful things", and they have done their part; but the inspired, scientific minds which conceived these great things and the constructive, courageous minds which executed these great things are superior to those instruments which they have employed.

What an epic of engineering and human progress has made possible our civilization and our Country with greater liberty, greater opportunity, and greater happiness to a greater number than has come anywhere at any time in the World's history.

Defenders of Our Country

Then came the World War with its years of bloodshed and sacrifice. Our civilization had been established and it must be defended. Mindful of the past and thoughtful of our future, we went forth, to protect and preserve our Country, its ideals and institutions, wrought from a wilderness after 140 years of superhuman effort by a great, a courageous, a patriotic, and a noble people, and we went forth to protect and preserve a world from chaos.

The Founder Societies, Engineering Council, National Research Council, civilian and commissioned engineers in all branches of transportation and industry, rendered indispensable service in the World War, but their activities were too diversified to be treated separately here. I shall speak of the Construction Division of the Army, which was a great human institution having only six Regular Army men in its entire organization and functioning under the single idea of emergency construction for war purposes; an assemblage of engineering and business skill of unusual and extraordinary capacities, consisting of chief engineers, consulting engineers, assistant engineers, water and sewerage experts, construction superintendents, and business men, supported by the War Industries Board with the vast resources at its command and ably led by Brigadier-Generals Littell and Marshall.

The Committee on Emergency Construction and Engineering of the War Industries Board, with only advisory powers, was formed in early May, 1917,* and its immediate problem was the housing of 1 344 000 men by September of that year. Before contracting any project, the General Staff requested and recommended it, the Emergency Construction Committee of the War Industries Board recommended it and recommended a contractor with an alternate. The Director of Purchases approved the recommendation, and the Construction Division, if it concurred, had to obtain the Secretary of War's approval of the project and the contractor and obtain his authority to proceed with the construction. Under such regulations and restrictions, the possibility of favoritism in the selection of the best, available, qualified contractor and the possibility or probability of collusion or conspiracy between five independent Departments and Bureaus, to defraud our Government in the great crisis, is too remote for an informed or a straight-thinking mind to believe and as an Officer in the Construction Division of the Army, I was in an advantageous position to know.

The Construction Division of the Army, with its 1 429 commissioned officers in charge of 30 000 clerical employees and a maximum of 427 000 laborers and mechanics, was an independent Staff Corps, without relation to the Quartermaster's Corps nor to the Corps of Engineers. Prior to the World War, with three officers and thirty civilians, Colonel I. W. Littell was in charge of the construction and repair work at the Regular Army Posts, which quartered 100 000 men. Upon his retirement as Brigadier General in 1917, after having worn himself out leading his organization through the successful completion of the first cantonment program he was superseded by Brigadier-General R. C. Marshall, Jr., M. Am. Soc. C. E.

On April 6, 1917, war was declared and on April 12, 1917, the Secretary of War declared that an emergency existed within the meaning of the "Revised Statutes and other Statutes", which exempted war emergency expenditures from the necessity of advertising for bids, and the Secretary ordered that until further instructed such contracts would be so made. The immediate problem confronting the Government was the construction of sixteen National Army Camps within a period of three months for the housing and training of 660 000 men. Before the construction of these sixteen camps was well under way, the Division was directed to construct within two months sixteen additional tent camps for the National Guard with a capacity for 684 000 men, and to be ready for the first contingent in two weeks. The civilian engineers and construction leaders of the country realized that a staggering program confronted the Nation for the housing and training of men and for the construction of innumerable other war projects on which the destinies of the overseas forces must depend.

In addition to the construction of the thirty-two National Army and National Guard Camps, this vast building program involved the construction

* This Committee consisted of William A. Starrett, M. Am. Soc. C. E., *Chairman*, M. C. Tuttle, transferred to the Emergency Fleet Corporation in 1918 and replaced by Major Clair Foster, C. W. Lundoff, Assoc. M. Am. Soc. C. E., Frederick Law Olmstead, and William Kelly, Major, Corps of Engineers, U. S. A., M. Am. Soc. C. E., later transferred overseas and replaced by John Donlin, representing the American Federation of Labor, with the late Leonard Metcalf and George W. Fuller, Members, Am. Soc. C. E., constituting the Sub-Committee on Engineering.

of hospitals with 121 000 beds; of terminal ports requiring roads, railroads, docks, wharves, and piers; of interior storage warehouses at industrial centers; of proving grounds, arsenals, and plants for the manufacture and storage of powder, high explosives, acids, and gases; of flying fields for the Signal Corps and Division of Military Aeronautics; of mechanical repair shops for assembling and repairing of automobiles and trucks in connection with the mobile artillery for the Motor Transport Corps; and of embarkation facilities for the Transport Service; with numerous other projects of lesser magnitude.

The camp and building sites for these projects had not been selected, the topography of the ground was not known, the extent of the projects had not been determined. Sewerage, drainage, and water supply had to be provided and typical buildings had to be designed that could be adapted to the sites when selected. It was at once evident to the War Industries Board that a form of contract must be devised that would expedite construction and safeguard the Government's interest; that a supervisory, administrative Governmental organization must be set up; and that a nation-wide survey of the construction industry must be made in order to obtain the most competent organizations in the construction fields, as no department of the Government had any adequate, up-to-date information on the construction industry.

Within two weeks the War Industries Board had secured reliable information on 1 500 contracting companies in the United States, based on information from engineers, architects, and the contractors themselves, and nationally known civilian engineers and experienced men were summoned by wire to fill the important posts in the Emergency Organization.

Under the emergency form of contract, the Government was enabled to proceed with its building program before details were completed, to push the work at any speed desired on any project great or small, to change the plan or scope of the work at will, and to get it done for such a reasonable fee as to be beyond the criticism of any informed individual.

After the thirty-two camps and cantonments had been completed and the advisability of concentrating construction work under a single head had been demonstrated, the Secretary of War on October 5, 1917, ordered that all construction for every Bureau and Division of the War Department within the United States, would be done by the Construction Division. This involved a total cost of $1 200 000 000 for 581 separate projects, located in every State of the Union except Arizona. It was made possible by an emergency construction program based on three considerations—the emergency form of contract that protected the Government's interest and was fair; the Construction Division organization that acted as administrators; and the competent, experienced contracting organizations that served as executors.

Under the emergency form of contract the contractor was allowed the actual cost at the site of the project with no "overhead", to which was added a sliding scale fee, depending on the magnitude of the job. On a contract estimated at $100 000, or less, the fee was based on 10% of the estimated cost, the maximum upset fee in all cases being fixed in advance by the contract. On a project estimated at $10 000 000 his fee was based on 2½% of the estimated cost, and on a project estimated at $25 000 000 (of which there were several),

his fee was based on 1% of the estimated cost, with a maximum upset fee fixed in advance by the contract.

In March, 1918, the Secretary of War appointed a Committee* to report on the methods of executing emergency work being done under the Construction Division. After careful consideration of the purchase and hire method, the lump sum contract, and the agency form of contract, the Committee reported unanimously in favor of the contract known as "cost of the work plus a sliding scale percentage with a maximum up-set fee" and further recommended that existing construction organizations be used for executing the work.

The complications in the construction industry arising from war conditions cannot be better illustrated than in the unfortunate experiences of the subway contractors of New York City. In the years 1914 to 1916, inclusive, 31 subway contracts were awarded on a unit price basis totaling \$125 000 000. When war was declared, there remained \$40 000 000 of unfinished work on these contracts. Nine suits for increased war costs on these contracts have been sustained and settlements have been made for \$5 100 000, and six other suits are now pending (July, 1928). Out of twenty of these subway contractors who became heavily involved in losses incident to the war, eight are still engaged in construction work, seven have never recovered from their financial losses, and the other five have gone out of business.

It must be evident from such a record of losses and failures on these unit price subway contracts, due to increased war costs, that either the Construction Industry would have been wrecked or the Government would have been compelled to pay prices out of all reason had unit price contracts been adopted for the war emergency construction.

Each camp was a self-contained community with every modern facility for the comfort and health of the men and Camp Grant, at Rockford, Ill., costing \$11 000 000 was typical. It had 1 600 separate buildings for quartering 43 000 men and 12 000 animals, with a hospital for 1 000 beds, and was ready to receive the incoming draft forces within $2\frac{1}{3}$ months after construction was started. As fast as these camp facilities were completed, the Utilities Section of the Construction Division took charge of the upkeep and operation of the heating and lighting plants, the sewer and water systems, and the roads. Thus, the Army entered and occupied a completed city in working order, with an adequate fire protection system.

Provisions were made for supplying each of our men with 50 gal. of water per day and the animals with 15 gal., which exceeded the British, Belgian, Canadian, French, and German supplies by 80 per cent. The greatest care and sanitary precautions were exercised in the disposal of sewage and wastes by modern methods, so that not a single case of typhoid fever or other water-borne disease was traced by the Medical Corps to impurities of the camp water

* This Committee consisted of: A. N. Talbot, *Chairman*, then President, Am. Soc. C. E.; Charles T. Main, President, Am. Soc. Mech. Engrs.; E. W. Rice, President, Am. Inst. E. E.; John Laurence Mauran, President, Am. Inst. of Archts.; R. G. Rhett, President, Chamber of Commerce of the United States; Frederick L. Cranford, President, Associated Gen. Contrs. of America, Inc.; John R. Alpine, Representative, Am. Federation of Labor; and Oscar A. Reum, Representative, Bldg., Constr. Employers Assoc.

supplies. As evidence of the efficient measures taken for proper drainage, comfortable housing, adequate hospitals, mosquito control, sewage disposal, and pure water, it is noted in the Mexican War that the deaths from disease amounted to 110 per thousand men; in our Civil War, to 65 per thousand; and in the Spanish-American War to 26 per thousand. In July and August, 1918, when the camps and cantonments were in full operation, the death rate from disease was 2.8 per thousand, which was one-fortieth that of the Mexican War, less than one-twentieth that of the Civil War, one-tenth that of the Spanish-American War, and less than one-half the death rate for men of military age in civilian life. This was a remarkable record of splendid service by our sanitary and water supply engineers in the greatest emergency this country has ever known.

Hospitals to the number of 294 were provided at a cost of $128 000 000, with accommodations for 121 000 patients, 12 000 nurses, 4 000 doctors, and 34 000 enlisted men. For the Ordnance Department, 60 projects costing $175 000 000 were constructed, consisting of arsenals, ammunition storage plants, gas plants, industrial plants, and proving grounds. At Aberdeen, Md., the proving grounds had a plant for assembling gun carriages and artillery and the largest machine shop in the United States. The reservation contained 35 000 acres, with facilities for every conceivable test for guns and munitions; a bombing field, and a field of 4 000 acres with equipment and plant for the manufacture and testing of gas shells, which was known as the Edgewood Arsenal and cost $30 000 000. There were quarters at this plant for 12 000 men, with a gas-filling plant having a capacity of 120 000 loaded shells per day and facilities for the production of caustic soda, mustard gas, phosgene, and picric acid, on an enormous scale.

Among the larger projects built by the Construction Division and representing the best types of terminal port facilities, were the seven Army Supply Bases at Boston, Mass., Brooklyn, N. Y., Port Newark, N. J., Philadelphia, Pa., Norfolk, Va., Charleston, S. C., and New Orleans, La., at which were collected the vast stores for overseas shipment. They were connected by rail to the fourteen interior warehouses at industrial centers, such as Philadelphia, Pa., Baltimore, Md., New Cumberland, Pa., Pittsburgh, Pa., Schenectady, N. Y., Columbus, Ohio, Chicago, Ill., St. Louis, Mo., and Jeffersonville, Ind.

These Army Supply Bases were combinations of railroad terminal facilities, storehouses, and piers for transhipment by ocean-going vessels. The Brooklyn Army Base, of the multiple-story type and the largest one built, has three 8-story concrete warehouses connected with one open dock and three double deck piers, which provide loading or unloading berths for twelve 8 000-ton ships at a time, with the necessary railroad tracks and yards to facilitate operations. This project was completed in fifteen months at a cost of $30 000 000, and has a total net floor area of 113 acres. These seven Terminal Ports and fourteen interior warehouses have a combined storage floor space of 631 acres with an additional open-shed area of 50 acres. The docking space at the seven Terminal Ports is sufficient to berth 65 ships of 8 000-ton capacity at one time. Eight hundred and forty-four miles of railroad tracks and 1 061 miles of concrete roads serve these 21 projects, the total cost of which was $234 000 000.

In July, 1918, the War Department appointed a "Board of Review of Construction"* to advise on the methods and procedure for carrying on the war construction program and to investigate and report on the work which had already been done by the Construction Division and by all other Bureaus of the War Department. The Board worked for more than a year with its own engineers and assistants, had free access to Government and contractors' records, and visited personally more than fifty construction projects. It reported as follows:

"The Board finds that the use of this form of contract, as finally developed, was well justified and contributed to the success of the emergency program; that by its use, speed was obtained in war construction projects; and that it is probable that such work could not have been performed in the time available without it or its equivalent.

"The Board of Review endorses and commends the action of the War Department in placing its construction work under one Bureau, entirely separated from the combatant units of the army, conducted with a minimum of military control, according to modern business methods. The Board is of the opinion that such construction so placed was done with a remarkable speed, was superior in quality, was characterized by economy of design, and was as economically performed as the requirements of speed and other war conditions permitted. The facts ascertained and given in the accompanying report indicate that such construction performance contributed materially to the success of the army operations."

How well these engineers and constructors carried out a great emergency construction program, is set forth in the following extracts from the Secretary of War's Reports of November and December, 1917 and 1918, to the President of the United States:

"The assembling of an organization and the planning and execution of the work was undertaken with a view of accomplishing all that human ingenuity, engineering and construction skill could devise in the brief time available and involved construction work which was in itself, in view of the time limitation, an almost staggering task."

* * * * * * * * *

"The preparation of places for the training of the recruits thus brought into the service, was a task of unparalleled magnitude."

* * * * * * * * *

"In spite of the stupendous difficulties involved, the entire housing enterprise was completed on schedule time, consisting of one of the most remarkable accomplishments of the war."

Despoilers of Our Country

After all this the heart of our nation was saddened and our thoughts were mingled with hatred at the excesses and the indecencies of some of our politicians, the greatest menace to our Country and the most destructive agencies of our institutions.

It became politically expedient to discredit the War Administration, so Congress appointed the Graham Committee to investigate war expenditures. Three thousand pages of *ex parte* testimony were exhibited for public consump-

* This Board consisted of Francis G. Blossom, M. Am. Soc. C. E., of Sanderson and Porter, W. Saunders Davies, President, Am. Inst. of Accountants, and Charles A. Morse, M. Am. Soc. C. E., President, Am. Ry. Eng. Assoc. and Chf. Engr., C., R. I. & P. Ry.

tion at the October Criminal Term of the Supreme Court of the District of Columbia in 1922, and the Assistant Secretary of War, with the Chairman and five members of the Committee on Emergency Construction and Engineering, were indicted for conspiracy to defraud our Country in carrying out its war construction program. To quote in part from the Jury findings:

"* * * said conspirators procured said work to be done at great expense to the United States by civilian engineers and their assistants, and consulting engineers and town planners, selected by said conspirators at large fees and great expenditure of the public funds of the United States, the aggregate amount thereof to the grand jurors unknown, but which they charge was in excess of one million dollars."

Relating to the findings of the Talbot Committee:

"* * * At Washington, D. C., in said District, in aid of inducing and procuring the continuance of the system of cost-plus contracts, awarded without competitive bidding, said defendant, William A. Starrett, about March 12, 1918, named and procured to be appointed by defendant, Benedict Crowell, as Acting Secretary of War, a committee, of which committee, certain members were interested in such contracts and system, ostensibly to investigate and report on the merits of said plan and system, which committee assembled on March 14, 1918, and reported on March 15, 1918, by adopting substantially as their report extended written statements and arguments made to them by said defendant, William A. Starrett and one Richard C. Marshall, Jr., then at the head of the Construction Division of the Army; to which report, as though it were a report of a thorough and impartial investigation, wide publicity was given."

Relating to the report of the Board of Review of Construction:

"* * * At Washington, D. C., in said District, about June 24, 1918, regardless of the interests of the United States, and in aid of procuring the indefinite continuance of the use of the cost-plus system of contracts without competitive bidding, said defendants, William A. Starrett and Clair Foster named and procured to be appointed by defendants, Benedict Crowell, as Assistant Secretary of War, a second committee composed of persons interested in said plan and system, and said conspirators then procured wide publicity to be given to such committee's report, as though said report was that of a disinterested and impartial investigation."

The Defendants entered demurrers to these criminal indictments and were sustained by Justice Hoehling, of the Supreme Court of the District of Columbia, in the following ruling:

"Accordingly, the Court is of the opinion, and so orders, that the demurrers filed herein be, and the same hereby severally are, sustained."

Civil suits amounting to $55 000 000 were brought against eleven of these cantonment contractors in 1922 by the Attorney General and, later, an additional suit was instituted, all charging a waste of public funds with the result that the men who built these cantonments, were without justification virtually and publicly branded as traitors. Many technical societies and similar organizations followed the lead of our Society and passed resolutions demanding of the Attorney General that he prove his allegations.

Primed with political chicanery and lacking in bills of particulars attempting to show how public funds had been squandered, only one of these suits ever

reached a jury. Within 3 min. after the Court's charges were concluded, the Jury returned a verdict for the Defendant. The Attorney General did not subpœna the former responsible officers of the Government in his attempt to prove his charges and the following former officers appeared for the Defendant: The Secretary of War; the Chief of the Construction Division of the Army; the Chairman and Consulting Engineer of the Committee on Emergency Construction and Engineering, and the Construction Quartermaster; the Supervising Engineer and the Supervising Auditor on the job.

Yet it was ten years after the cantonments were completed and five years after these suits were instituted when the administrator of the Department of Justice finally took action to dismiss the remaining eleven suits.

The criminal indictments of the Assistant Secretary of War and of the Committee on Emergency Construction and Engineering, the clearly implied charges of conspiracy against our outstanding and nationally known engineers, and the suits instituted against twelve of our competent cantonment contractors, indispensable in turning the tide of war, were brazen exhibits of political indecency and a slander on the patriotism of our people. The toleration of such conditions is a menace to our Country.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE ISTHMIAN CANAL SITUATION*

By Hans Kramer,† Assoc. M. Am. Soc. C. E.

Synopsis

General interest in needs for additional Isthmian canal facilities has been actively renewed. The factors which limit the capacity of the Panama Canal and the criteria upon which a forecast of the probable traffic increase is based, form the basis for a graphical method for predicting the saturation point of the existing installation. The conclusion is reached that the Panama Canal (augmented by the pending Alhajuela Reservoir development) will be adequate to handle all probable traffic until 1970.

The choice for additional facilities to be provided to meet future traffic needs is limited to the construction of additional locks at Panama and the opening of a new canal route *via* Nicaragua. The relative advantages and disadvantages of these alternatives are discussed from the present-day viewpoint. From this comparison, the installation of more locks at Panama is the logical method for supplying future needs for additional Isthmian canal facilities.

Neither the necessity nor advisability of a Nicaragua Canal appears to exist at present.

Newspapers, popular periodicals, and technical journals have contained numerous references in recent months to the impending inadequacy of the Panama Canal and to the necessity of additional Isthmian canal facilities, the latter preferably by way of Nicaragua. The agitation by feature writers and editors has been augmented by the public utterances and recommendations of Congressmen and public officials.

Note.—Discussion on this paper will be closed in December, 1928.

*This paper was written in May, 1927, and, therefore, is based on data obtainable at that time. Since then additional statistics and other information have come to light and have been added. Inasmuch as this study presents, in part, a prediction as of May, 1927, it has not been deemed desirable to modify the original manuscript by including developments of a later date.

† First Lieut., Corps of Engrs., U. S. A.; Asst. U. S. Dist. Engr., Philadelphia, Pa.

In view of the sudden revival of public interest in a problem which has enjoyed a well-earned rest for more than a decade, it seems pertinent to raise several fundamental questions on which the present and any future discussion properly ought to be based. First, when will the maximum capacity of the Panama Canal probably be reached? Second, what additional facilities will then be advisable? In general, the various recent statements have been predicated on the hypothesis that the capacity of the Panama Canal has already been reached, or that it will be reached in the very near future, and that, therefore, the second question of additional canal facilities is of immediate importance.

It is the purpose of this paper to develop the answers to these basic questions utilizing cold-blooded statistics, established facts, and the opinions of recognized experts. Except where necessary for this purpose, no descriptive matter will be included; the written works of the builders of the Panama Canal have incomparably covered this aspect. A general familiarity with the existing canal is, therefore, assumed.

Forecast—Time of Reaching Capacity

When will the maximum capacity of the Panama Canal probably be reached?

Varying arguments have been introduced to show the necessity for additional Isthmian canal facilities. Some of these have been directly stated, others only implied. All the pertinent points, however, can be readily classified. It seems sufficient to group these arguments in two main categories—technical reasons and non-technical reasons. The latter designation is used for convenience and is not intended to be derogatory. In fact, some of these non-technical considerations may be of greater importance than the technical ones.

Technical and Non-Technical Arguments.—The technical arguments that should receive primary attention are those that portend an early inadequacy of the present Panama Canal to handle the traffic offered. Such inadequacy would be evidenced by congestion due to excessive quantity of traffic or by inability to handle vessels of excessive dimensions.

The leading argument in the non-technical group is linked with the question of National defense. The American fleet has been taken expeditiously through the Panama Canal several times. A similar canal at Nicaragua, nearer Mexico and nearer the United States, with new navel bases as an integral part of the development, would undoubtedly add to the efficacy of these naval forces. But is this necessary? The opening of new Central American markets and outlets for Central American products by way of a Nicaraguan Canal is another non-technical argument (emanating undoubtedly from political, business, and banking interests), for early commencement of canal construction in Nicaragua. Out of this argument grows another—that such an enterprise would stimulate business through the circulation of a large part of the surplus of the United States Treasury. The proponents of this particular argument may also have in mind the utilization of the Treasury's selfsame

surplus to meet deficits during the long lean years before another canal becomes self-sustaining.

Some statesmen are probably hoping to make political capital out of their alertness and sagacity in being among the first to urge the construction of another or a bigger Isthmian canal. Others—perhaps more sincere, more altruistic, more philanthropic—see in a new canal *via* Nicaragua, not only the solution of the present vexing Nicaraguan question, but the settlement of the whole Central American political and economic problem for all time. All these are grouped as non-technical reasons to show the necessity for additional canal facilities.

Present Operating Capacity.—In approaching the basic question as to just when the capacity of the existing canal will be reached, it is relevant to ask the preliminary question: What is the capacity of the present canal and how is this capacity determined and limited? Concise and authoritative data on this question are given* by M. L. Walker, Brig. Gen., U. S. A., M. Am. Soc. C. E., Governor of the Panama Canal, which statements form the basis of the present conclusions, and, therefore, require no further proof for their authenticity and reliability.

The limiting factors at Panama are the operating speed of the locks, their dimensions, and the water supply for maintaining the level of the summit lake. Without going into the details of mathematical computations of speed of lock operation, suffice it to say that the original designers conservatively estimated† that the Panama Canal would be able to handle 80 000 000 net tons of shipping per year. This estimate contemplated 24-hour per day operation, allowing ample shut-downs for periodic repair and overhaul. Actual experience has borne out the correctness of this estimated maximum tonnage capacity.

Each lock at Panama has usable clearances of 1 000 ft. of length, 110 ft. of width, and 40 ft. of depth. For commercial purposes these dimensions are ample. The general tendency in the design of commercial vessels shows a departure from the *Leviathan* type for reasons that are not essential to this discussion. The Washington Conference placed a "damper" on competitive naval armament and the consequent development of super-dreadnought sizes. Had the pre-war rate of growth of battleship dimensions continued without abatement the limiting size of the Panama locks to-day would be a serious consideration. Therefore, the construction of both commercial and naval ships of such a size as would prevent their transit appears at present to be only a remote possibility.

Water Supply as a Limiting Factor.—The other physical factor affecting the capacity of the Panama Canal is the water supply for maintaining the level of Lake Gatun at a height to afford ample depth of channel through the summit section of the canal. As is well known the rainfall in that drainage area is not ideally distributed throughout the year. The supply is ample in gross quantity, but is so distributed as to provide an excess of water during the rainy season and, generally, a shortage during the four months of the dry

* "The Panama Canal After Ten Years", *The Military Engineer*, May-June, 1925.
† "The Panama Canal," by Messrs. Bakenhus, Knapp, and Johnson.

season. In other words, the result is that the maximum amount of traffic can always be accommodated during the rainy season—water is sufficient for all lockages—but it may be limited during the dry months. It has been estimated reliably that with even the dryest of dry seasons, the existing canal facilities can accommodate from 30 000 000 to 40 000 000 tons of shipping per year without interruption of service.*

Contrary to current popular belief, this limitation is not a new discovery, nor an evidence of improper design and lack of forethought on the part of the builders. Neither is it a serious menace to the continued operation of the canal under any conditions of traffic which have been encountered to date. The characteristics of the rainfall in the Lake Gatun area were well known and fully considered when the canal was designed. It was realized that construction of an additional dam to create another storage reservoir in the Chagres River Valley above Lake Gatun would be necessary before the ultimate capacity of the canal as determined by the maximum number of lockages could be utilized. Accordingly, hydrological and other data have been compiled for years in anticipation of the inevitable dam and reservoir construction at Alhajuela, Panama, just outside (east of) the Canal Zone.

The expected traffic in the early years of operation did not require the additional storage basin for maintaining the lake level, so that this development was very wisely left for later years. Since annual traffic is now approaching the expected stage, and since no reasons exist for believing that the Panama seasons will change their rainfall characteristics, it is now advisable and justifiable to appropriate funds for the long considered Alhajuela Dam construction.† Other benefits, such as hydro-electric power, would also result from this project, which involves no particularly difficult engineering problems. Four or five years' time for construction and an expenditure of $10 000 000 to $15 000 000 will insure the continued operation of the canal under the maximum traffic load of 80 000 000 tons annually, regardless of the fluctuations of the rainfall.‡

Records of Panama Traffic.—Having analyzed the physical factors which limit the capacity of the Panama Canal, the next logical step is to review the history of canal traffic. Table 1, from the official records of the Panama Canal, shows the amount of shipping in net tons through the canal during each calendar year since its opening. These data do not include U. S. Naval vessels and miscellaneous craft exempt from tolls. It is obvious that, during the year of heaviest traffic to date (1927), the 26 600 000 tons taxed the canal capacity to about one-third of its maximum of 80 000 000 tons per year. This fact is entirely reconcilable with the operating policy of the canal, that is, to receive ships for transit during only 8 hours of the 24-hour day. In other

* "The Construction of the Panama Canal," by Sibert and Stevens.

† The 1929 Army Appropriation Bill, H. R. 10 286, approved March 23, 1928, contains the following item:

"$250 000 for commencing the construction of a dam across the Chagres River at Alhajuela for the storage of water for use in the maintenance and operation of the Panama Canal, together with a hydro-electric plant, roadways, and such other work as in the judgment of the Governor of Panama Canal may be necessary, to cost in the aggregate not to exceed $12 000 000."

‡ "The Panama Canal," by H. F. Hodges, Maj. Gen., U. S. A. (*Retired*), M. Am. Soc. C. E., *Professional Memoirs,* U. S. Corps of Engrs., October–December, 1909.

words, the canal is regularly operating at full speed and to maximum capacity during the one-third of the time traffic is accepted.

TABLE 1.—PANAMA CANAL TRAFFIC.

Calendar year.	Net tonnage.
1914*	1 284 293
1915†	3 902 592
1916†	3 817 704
1917	6 217 054
1918	6 409 886
1919	6 943 087
1920	10 378 265
1921	11 435 811
1922	12 992 573
1923	24 737 437
1924	24 411 760
1925	22 958 158
1926	25 836 241
1927	28 610 984

* Canal opened August 15, 1914.

† Canal closed three months by slides.

In addition, during some of the heaviest months of commercial traffic (and during the dry season as well) the bulk of the American fleet has been expeditiously passed through the canal without interfering with routine commercial traffic, simply by increasing the number of operating hours. Regularly the locks have been placed out of commission for overhaul and repair. Yet actually the minimum lake level has never been reached. The Panama Canal has operated and is operating regularly and comfortably at less than one-third the ultimate capacity of the present installation of locks. Use of less than one-third capacity, therefore, can hardly be termed a congested condition nor a situation which presages early inadequacy of the existing facilities.

Comparison of Suez Traffic.—The final step in foretelling the exhaustion of surplus capacity lies in predicting the future from the past and present. Dr. Emory R. Johnson, Dean of the Wharton School of Commerce and Finance of the University of Pennsylvania, who was a member of the original Isthmian Canal Commission, faced a similar problem in the preparation of his report on Panama Canal traffic and tolls as Special Commissioner for the President in 1912. Profiting from his experience and personal advice, two different but related criteria are introduced to study the probable future trend and growth of canal traffic. The first is the history of traffic through the Suez Canal, which is of much longer record than Panama and is singularly similar as an artificial highway of international shipping. The other basis is that of world commerce, or the combined exports and imports of the principal nations of the globe, which furnishes a fairly reliable barometer of the shipping through both Suez and Panama.

The annual Suez Canal traffic from the opening, and the annual total value of the world's commerce in billions of dollars for the same period are given in Table 2. The Suez statistics are taken from the official Suez Canal Bulletin. The source of the world commerce figures is the Statistical Abstract compiled annually by the U. S. Department of Commerce.

The statistics of Tables 1 and 2 (Panama traffic, Suez traffic, and world commerce) are charted and presented graphically in Fig 1. The salient points should be noted—the apparent smoothness and gradual upward increase of Suez traffic (Curve *A*) from the opening of the canal in 1870 until the beginning of the World War in 1914; the world commerce curve (Curve *B*), similarly smooth and gradually increasing over the same period of time, although of course it does not originate at zero in 1870; the terrific jar to the world's equilibrium caused by the World War and reflected by the extreme peaks and valleys in the curve between 1914 and 1918; the rapid and steady climb in Panama Canal traffic during the canal's first decade of operation, as evidenced by the steep general slope of the Panama curve (Curve *C*); finally, and interestingly, too, the Panama traffic in 1926, practically equal to the Suez traffic. The vertical scale for charting world commerce having been suitably chosen, all three of the curves under consideration virtually attain a common point in 1926, which, in turn, permits of starting at a common origin in projecting the curves into the future.

TABLE 2.—Suez Canal Traffic and World Commerce.

Year.	Suez Canal, in net tons.	World commerce, in million dollars.	Year.	Suez Canal, in net tons.	World commerce, in million dollars.
1870	436 609	$10 633	1899	9 895 630	......
1871	761 467	......	1900	9 738 152	$20 105
1872	1 160 743	......	1901	10 823 840	......
1873	1 367 768	......	1902	11 248 413	......
1874	1 631 650	......	1903	11 907 288	......
1875	2 009 984	......	1904	13 401 835	......
1876	2 096 772	......	1905	13 134 105	......
1877	2 355 448	......	1906	13 445 504	27 418
1878	2 269 678	......	1907	14 728 434	......
1879	2 263 332	......	1908	13 633 283	......
1880	3 057 421	......	1909	15 407 527	......
1881	4 136 780	14 761	1910	16 581 898	33 364
1882	5 074 804	......	1911	18 324 794	35 909
1883	5 775 862	......	1912	20 275 120	39 750
1884	5 871 501	......	1913	20 033 884	40 420
1885	6 335 752	......	1914	19 409 495	37 760
1886	5 767 656	......	1915	15 266 155	31 302
1887	5 903 024	......	1916	12 325 347	46 523
1888	6 640 834	......	1917	8 368 918	52 781
1889	6 783 187	......	1918	9 251 601	62 802
1890	6 890 094	......	1919	16 013 802	75 311
1891	8 698 777	17 519	1920	17 574 657	61 277
1892	7 712 029	......	1921	18 118 999	61 417
1893	7 659 068	......	1922	20 743 245	46 137
1894	8 039 175	......	1923	22 730 162	49 918
1895	8 448 383	......	1924	25 109 882	50 732
1896	8 560 284	......	1925	26 761 935	57 608
1897	7 899 374	......	1926	26 060 377	55 000*
1898	9 238 603	......	1927	28 965 000	......

* Estimated; complete official figures for 1926 not yet available.

Whereas Fig. 1 is an authentic graphic portrayal of official statistics, Fig. 2 is partly authentic, partly artificially manipulated, and partly conjectural. The Suez traffic (Curve *A*) and the world commerce curve, (Curve *B*) are true copies from Curves *A* and *B* of Fig. 1 for the periods 1870 to 1913 and 1921 to 1926. The World War period, including the years of restoration to normalcy, 1914 to 1920, has been arbitrarily and artificially effaced from Fig. 2 by the simple mechanical expedient of plotting the year

1921 to follow immediately after the year 1913. For convenience a new curve, Curve *D*, has been introduced representing Suez Canal traffic averaged by decades. This smoothes out some of the minor irregularities in the Suez yearly curve. The last decade for Curve *D* also eliminates the World War period and averages the annual traffic for the ten years, 1910 to 1913 and 1921 to 1926, inclusive.

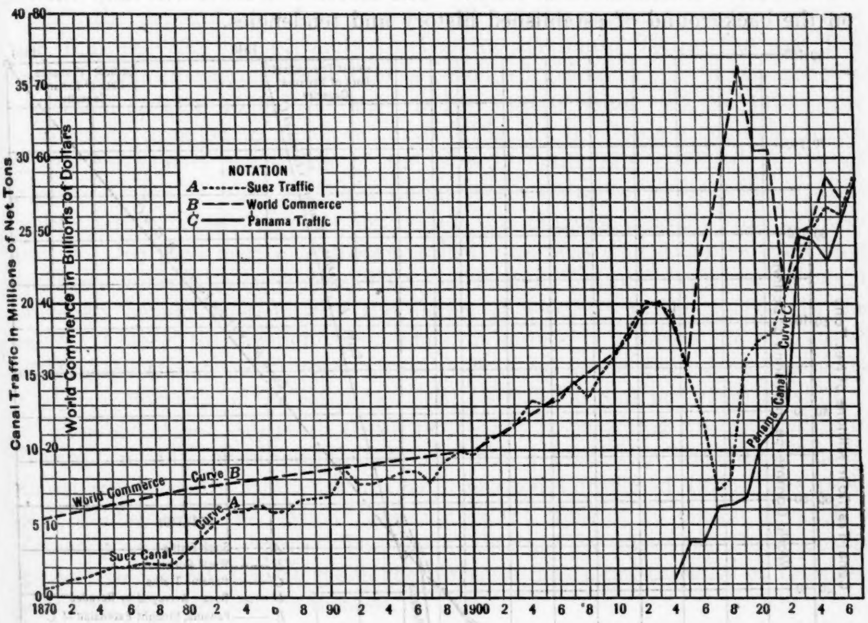

FIG. 1.

Future Trends.—Now comes the rub! The prolongation of a known curve into the unknown future is at once a dangerous and difficult undertaking, and as has often been caustically said, "figures won't lie but liars will figure". The extension of a curve beyond its known values is particularly precarious and its reliability consequently is subject to wide differences of opinion when the length of the projection considerably exceeds the length of the established record. Predicting future behavior over a 5 or 10-year period from a given record of 50 years usually can be done fairly safely. A similar projection of a much shorter curve has many pitfalls; and a 50-year prolongation of a 15-year curve is like shooting at the moon. It can be done, however, and in some respects it is less complicated than making a similar projection for the much shorter 5 or 10-year period.

In attempting to forecast for the next decade a thorough analysis of political, economic, and social conditions throughout the world would be necessary to determine their probable trend and corresponding probable effect on canal traffic. Temporary local disturbances and commercial fluctuations would have to be carefully studied in a prediction covering only the next few

years. However, in considering the next 50 to 100 years, unexpected, unknown, unpredictable factors may arise, such as world-wide disturbances, development of new trade centers and routes, revolutionizing inventions, etc., which events may entirely overshadow the effect of any expected, known, and predictable factors. In this prognosis of canal traffic over a relatively long time, therefore, no attempt is made to analyze or predict the conditions which will affect future Isthmian canal traffic. The forecast is made virtually entirely on the background of established history and tendencies.

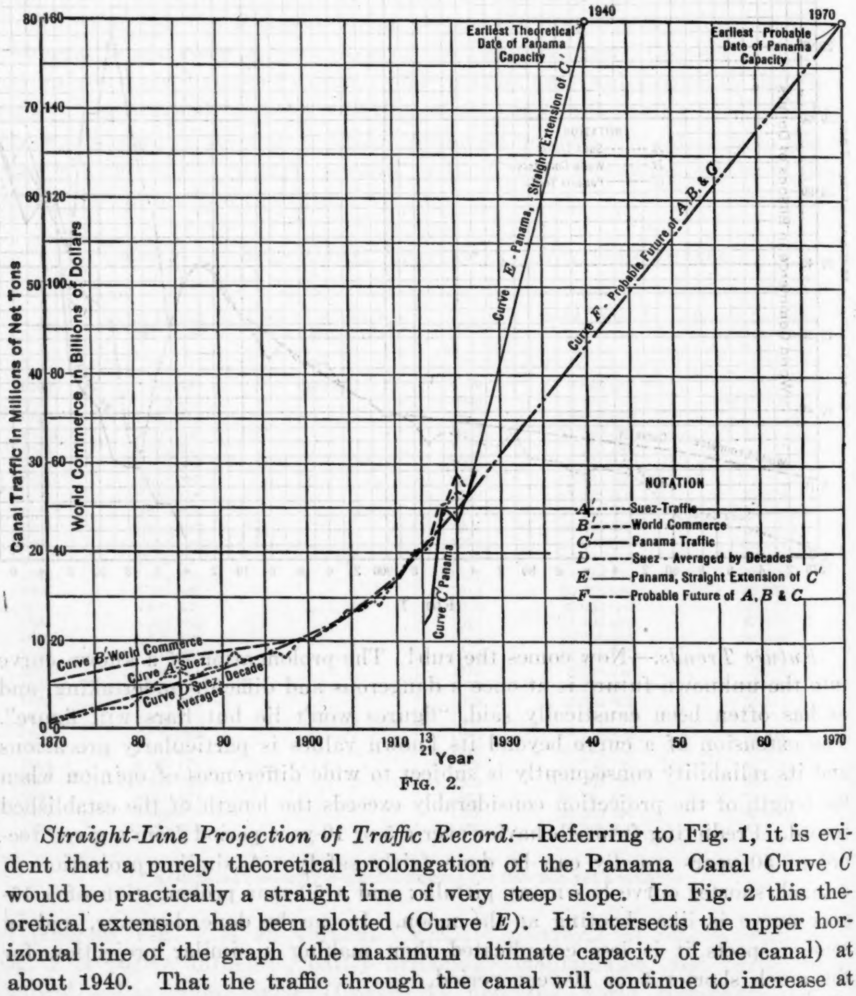

FIG. 2.

Straight-Line Projection of Traffic Record.—Referring to Fig. 1, it is evident that a purely theoretical prolongation of the Panama Canal Curve *C* would be practically a straight line of very steep slope. In Fig. 2 this theoretical extension has been plotted (Curve *E*). It intersects the upper horizontal line of the graph (the maximum ultimate capacity of the canal) at about 1940. That the traffic through the canal will continue to increase at its past rate so as to reach the saturation point in 1940 seems highly improbable. The creation of new devices usually results in rapid growth of utilization and in every healthy case such growth continues but generally at a more moderate rate after the novelty has worn off. Table 1, in fact, verifies this general law of behavior as applied to the Panama Canal, for the statistics of

traffic since 1923 indicate normal and steady conditions after the rapid increase of the first decade. This theoretical straight-line extension (Curve *E*) of the Panama Canal traffic curve can be interpreted as the extreme condition in growth of traffic which would result in reaching the maximum canal capacity within the next fifteen years, if the rate of increase during the first decade continues—a highly improbable condition.

A condition more likely to prevail over a long period of future growth is that the present annual increment will remain constant, on the assumption that the rate has probably reached its maximum during the past few years. Such an assumed condition is portrayed graphically by a straight-line projection tangent to the curve at its last known point. As the 1926 point is virtually common to all three curves (Fig. 1), it affords a common origin in the speculative projection of the already artificially modified curves of Fig. 2. Curve *D,* showing Suez Canal traffic averaged by decades, is arbitrarily selected for the construction of the tangent straight line, Projection *F*. No high degree of mathematical exactness or mechanical precision has been resorted to in constructing this tangent for such refinements would obviously be inconsistent with the degree of accuracy to be expected in the result. Line *F,* then, which represents simultaneously a theoretical prognosis* of Panama Canal traffic, Suez Canal traffic, and world commerce, intersects the saturation line of the Panama Canal (80 000 000 tons per year) at 1970.

Panama Canal Adequate Until 1970.—Let it be stated unequivocally, to forestall misinterpretation, that this is not a prediction that the Panama Canal will actually attain its ultimate maximum traffic in 1970. The problem of answering the "when" question is too difficult to permit of such a definitely accurate answer. The result obtained is, therefore, the probably propective earliest date at which the ultimate maximum capacity of the present canal facilities will be reached.

The 1970 intersection is the result of an assumed straight-line projection. That such a high rate of increase will continue uniformly for the next fifty years is, of course, possible, but it is also more than likely that while traffic will continue to increase, its rate of growth will decline. In other words, the curve of the future is very apt to reach a point of inflection where it will depart from its straight upward course and tend to become more nearly horizontal. This tendency can reasonably be expected in the higher range of the diagram, justifying the inference that the straight-line tangent projection is reasonably conservative in predicting growth over a long period of time. Had the projection been actually made on the true curves of Fig. 1, rather than on the artificial curves of Fig. 2, it is evident that the slope of Curve *F* would have been somewhat nearer the horizontal and there would have been a resultant later intersection with the upper saturation line. This further emphasizes the conservatism of the 1970 intersection.

If the accuracy of established records, the reasonableness of assumptions, and the conservatism of construction methods are acknowledged, then the result obtained should be accepted. It is believed that all these various fac-

* Traffic statistics for the calendar year 1927 are as follows:

Panama Canal..................	28 610 984 net tons
Suez Canal........................	28 965 000 net tons

tors have been satisfactorily explained and established. The conclusion therefore is warranted that the Panama Canal (augmented by the justifiable Alhajuela Reservoir development) will probably not be taxed to its maximum capacity within the next forty years, and that it will prove adequate to handle all probable traffic until the calendar reads 1970.

Provision for Additional Capacity

What additional canal facilities should be provided when the present installation reaches its capacity? This second question is as important as the first and follows it most logically.

Having deduced that the present canal is adequate to handle all probable traffic for at least forty years, it seems somewhat paradoxical at this time to open any discussion of additional Isthmian canal facilities. Even granting that such a discussion is relevant, it is obvious that any comparative analysis of possible expansion projects must be made as of 1927. Many of the factors entering into such an analysis may remain materially unchanged over a long period of time, but others may be markedly altered as to character and effect during the next forty years. Physical conditions may remain practically as they are, but political conditions, particularly in Central America, are constantly changing their complexion. Social and economic factors, too, are not permanent, but are likely to vary in aspect and influence from time to time. In the ensuing discussion, therefore, the unavoidable inconsistency of presenting the 1970 situation from the 1927 viewpoint must be kept in mind.

Alternative Possibilities.—However, let it be assumed, for the sake of argument, that the Alhajuela Dam has been built and that the Panama Canal is carrying its maximum load of traffic. What additional facilities are now feasible? In all present-day discussions, fortunately only two alternatives are recognized. All Isthmian canal routes other than by way of Panama or Nicaragua were eliminated from consideration many years ago. Either supplementary sets of locks can be installed at Panama or a new canal can be constructed through Nicaragua.

The expansion at Panama would involve the construction of a third and perhaps, also, a fourth set of locks paralleling the existing twin-lock installation. The effect of such improvement would be the creation of a highway accommodating three or four lines of traffic where only two can flow at present. This enlargement would naturally entail some minor modifications in the adjoining existing structures, but would not involve any duplications or major changes in the other canal essentials, namely, Gatun Lake, the summit pool, Gatun Dam which impounds the lake water, Culebra Cut, the "big ditch" through the Continental Divide, and the terminal harbor facilities at Cristobal and Balboa. With the concurrent development of all feasible plans for increased storage and conservation of water no apprehension need exist as to any probable dry-season water shortage. The Nicaraguan project involves complete new construction starting on the Atlantic side in the vicinity of Greytown or San Juan del Norte, canalizing the San Juan River along

the Nicaragua-Costa Rica border, through Lake Nicaragua, piercing the Continental Divide and emerging at the Pacific side in the vicinity of Brito.

Pros and Cons of Panama and Nicaragua.—The advantages and disadvantages of the Panama and Nicaragua routes were exhaustively studied by the Isthmian Canal Commission and other eminent authorities before the Panama Canal was built. In the present comparative study of the merits of the two practicable expansion projects it is not intended to repeat long extracts from the Commission's authoritative report nor to re-open the discussion of mooted questions which found their answers in the monumental accomplishments of the late George W. Goethals, M. Am. Soc. C. E., the late General Gorgas, and others. These records, written and physical, speak for themselves. For convenience the comparison of the feasible expansion projects at Panama and Nicaragua has been compiled in simple form in Table 3.

To accord with the classification of reasons in the discussion of the probable time of exhaustion of present facilities the comparative factors in Table 3 are grouped under technical and non-technical headings. The more important of the items will be elaborated. Although the factors listed undoubtedly do not exhaust the entire field, yet they are sufficiently complete for the present purposes. They are, in general, broad enough individually to embrace other incidental considerations and comprehensive enough collectively to cover the essential features.

TABLE 3.—COMPARISON OF EXPANSION PROJECTS.*

Item. (1)	Panama. (2)	Nicaragua. (3)
Technical:		
Type of canal	Lock (85 ft. lake level).	Lock (110 ft. lake level).
Length of canal	50 miles.	175 miles ±.
Terminal harbors	Excellent.	Pacific, fair; Atlantic, poor.
Navigation	Easy.	Difficult.
Length of steamship routes } offset.	Intercoast somewhat longer.	No advantage for South American trade.
Time of transit } offset.	6 to 8 hours.	24 to 30 hours.
Earthquakes	Not a serious factor.	More serious than Panama.
Construction facilities	Excellent existing.	None existing.
Additional capacity	40 000 000 to 75 000 000 tons (?).	80 000 000 tons (twin lock) (?).
Cost	$150 000 000 to $250 000 000 (?).	$500 000 000 to $1 000 000 000 ?.
Time to complete	8 to 10 years (?).	15 years (?).
Non-Technical:		
Defense—separate	Vulnerable.	Vulnerable.
Defense—simultaneous	Very difficult.	Very difficult.
Central American trade	No further effect.	Slight development.
Construction rights	Perpetual.	Perpetual.
Political situation	Stable.	Unstable.

* Corroborative evidence of reliability of the comparisons in Columns (2) and (3) can be found in the Official Report of the Isthmian Canal Commission, 1899–1901, and in "Problems of the Panama Canal," by Gen. Henry L. Abbot.

Comparison of Physical Features.—Frequent misstatements have been made regarding some of the inherent physical characteristics of the two routes. The Panama Canal, it is well known, is of the lock type with a nominal lake summit level of 85 ft. above the sea. The difference in elevation is overcome by twin flights of three locks at each end. Suitable foundations are known

to be available for the construction of a third set of locks and may exist for even a fourth set. From deep water to deep water at both ends the Panama Canal has a total length of 50 miles, of which 24 miles are through Lake Gatun and 8 miles through Culebra Cut.

The question of lock *versus* sea-level design was widely discussed during Panama Canal construction days; but it applied only to the Panama route. Newspapers and magazines to-day refer nonchalantly to the advantages of a lockless sea-level canal by way of Nicaragua. Such references are little short of absurd. Whereas, the Panama route did originally offer a choice between the sea-level and lock alternatives, the Nicaragua route is universally recognized by the Engineering Profession to be impracticable for anything but the lock type of construction.

Proposed plans for a canal *via* Nicaragua contemplate the utilization of the large existing inland body of water, Lake Nicaragua. This would necessitate the maintenance of a nominal summit lake level of approximately 110 ft. above the sea. At least three steps of locks, as at Panama, and probably four, would be required to negotiate this difference in elevation. The total length of the Nicaragua route is about 185 miles, of which about 100 miles would be along the crooked San Juan River which would require artificial canalization. About 70 miles would be through Lake Nicaragua itself which would include about 20 miles of channel dredged (and continuously maintained) through the soft mud and fine silt in the shallow southwestern part of the lake. The severing of the remaining obstacle, the Continental Divide near the Pacific end, would require a cut perhaps 20 miles in length. This brief summary of the outstanding physical features of the two routes is sufficient to indicate that the Nicaragua project would be an even more stupendous undertaking than the construction of the Panama Canal. This prodigiousness, of course, would be reflected in the ultimate cost of construction.

In comparing the item of terminal harbors Panama possesses a wide advantage. The existing harbors at Cristobal and Balboa are excellent examples of the most modern type of port development. At Nicaragua, two harbors remain to be created. The one at the Pacific end could probably be built at a reasonable cost, but the site on the Atlantic presents unusual natural difficulties both for construction and maintenance.

Relative Advantages for Navigation.—The difficulties to be met in actual navigation are an important consideration. The gentle curvature of the channel of the Panama route permits of easy negotiation. The Nicaragua channel would be much more tortuous. The relative lengths of the two channels, has a weighty bearing on the question of navigation difficulties, particularly in view of the necessity for night traffic. Much more severe rainfall than is prevalent in Panama is to be expected in Nicaragua. Then, too, the heavier trade winds and stronger channel currents will be encountered. The navigator's choice would undoubtedly favor Panama.

The most ardent advocates of the Nicaragua route, both past and present, propose, as one of their strongest arguments, the advantage of the Nicaragua route in shortening travel distances (and hence time) between principal ports.

The validity of this argument is, however, more apparent than real. Where differences in distance favoring the Nicaragua route do exist, the apparent advantage is fully compensated by the difference in time required to traverse the canals and the inevitable high difference in insurance rates caused by the difference in navigation risks. As compared to Panama the actual sea distance by way of Nicaragua is 666 statute miles shorter between San Francisco, Calif., and New Orleans, La., and 434 miles shorter between San Francisco and New York, N. Y., a gain in time of approximately 48 hours and 31 hours, respectively, for the average modern steamship. Between the eastern coast of the United States and the western coast of South America—a much frequented trade route—the apparant advantage of shorter distance by way of Nicaragua disappears entirely for the eastern terminus of the Nicaragua route is actually farther from the Atlantic seaboard than the eastern end of the Panama Canal.

The time required for passing through the canals must, of course, be considered concurrently with the relative sea distances. At Panama from 6 to 8 hours are consumed in traversing the Isthmus. Estimates for Nicaragua vary, but the time may be conservatively placed at from 30 to 36 hours without allowing for probable frequent delays caused by forced retarding of operating speeds. The closer comparison of the routes, therefore, on the basis of economy of vessel operation, shows that the merits of the Nicaragua route in this respect are generally over-emphasized.

Feasibility of Constructions.—The relative possible effect of earthquakes on structures and excavations along the two routes is not considered of primary importance. At Panama the slides are no longer a problem and earth tremors have never been serious enough to affect other structures. What the history of the Nicaragua route would be in this respect is difficult to divine. The fallacy of a sea-level waterway having been previously shown, the claims for any additional inherent stability and permanency of this type of construction are entirely vitiated. Judging from past records and natural existing conditions along the route a lock canal at Nicaragua would be extremely fortunate to enjoy as uninterrupted an existence as the one at Panama.

The influence of the utter lack of construction facilities, such as a paralleling railroad along the Nicaragua route, is obviously included in the item of cost of the project. It should be noted too that the availability of plant and exact knowledge of working conditions at Panama permit of a cost estimate with a fairly high degree of accuracy, whereas the absence of all such facilities and the uncertainty of working conditions necessarily make the Nicaragua estimates much less reliable.

Cost Estimates Favor Panama.—Although specific estimates have been made as to the comparative costs in time and money of the two alternatives, the bases are mysteries. Since the report of the Isthmian Canal Commission in 1901 no further official investigation has been made nor estimate prepared of the Nicaragua project. Neither have any recent official estimates been prepared and published for additional lock construction at Panama. Estimates in the engineering sense generally imply preliminary analyses

without which they are merely guesses. The currently quoted figures are undoubtedly merely offhand guesses which by repetition have been dignified by the appellation, "estimates".

The present lack of prepared estimates applies with equal force to both the Nicaragua and Panama expansion projects. The generally quoted figures are from \$150 000 000 to \$250 000 000 for additional lock construction at Panama and from \$500 000 000 to \$1 000 000 000 for a new canal *via* Nicaragua, with an estimated time of construction of 6 to 9 and 12 to 15 years, respectively. No mention is made of the additional tonnage capacity which will be provided by these expenditures. The amount of such increase will depend largely on the design adopted, but it may be presumed that both projects would provide about 100%, or 80 000 000 tons per year, additional capacity.

A simple attempt may be made to arrive at some estimates of cost slightly more reliable than mere guesses. The method is admittedly crude, but sufficiently satisfactory for the present rough comparative purposes. All details, some of which are of major importance, are purposely omitted from consideration. The report of the Isthmian Canal Commission in 1901, on which the decision between Panama and Nicaragua at that time was finally based, estimated the cost of the Panama project at \$144 000 000 and that of the Nicaragua project at \$190 000 000. As actually built the Panama Canal cost \$347 000 000. By simple proportion, taking into account the depreciation in the value of the dollar, the construction cost in Nicaragua in 1927 can be estimated at \$685 000 000. From the total cost of the Panama Canal it is difficult to segregate the cost of the locks as a separate item, but it approximated \$76 000 000. Again, by simple proportion, a 1927 duplication of the Panama locks can be estimated at \$125 000 000. Regardless of the absolute accuracy of these estimates they undoubtedly indicate the relative costs involved in choosing between the two alternatives.

A cursory summary of the advantages and disadvantages possessed by the two feasible expansion projects, as far as technical aspects are concerned, shows that the balance is decidedly in favor of the Panama alternative. From the unrelenting viewpoint of economics, dollars and cents, the Panama choice appears to be indisputable.

Non-Technical Arguments.—In the same way that factors other than technical have a bearing on the necessity for additional canal facilities, they must be given due consideration in comparing the feasible expansion projects. The order of their presentation is no indication of the relative importance of these items, for naturally their weight is neither constant nor commensurable.

Perhaps the most interesting of these items is the question of defense of the two routes. Being fundamentally of the same type of construction, it follows theoretically that in this respect the two alternatives will be equally vulnerable to hostile attack. The one presents perhaps a more concentrated target, the other a more extensive one. Both require fortification and defensive organization on a common scale.

The adage, "It is unwise to carry all your eggs in one basket", has been invoked to catch popular fancy as an argument for a Nicaragua Canal. The appropriateness of the proverb is questionable, however. The Isthmian canal situation is not a question of carrying eggs, but of protecting baskets. It has been found difficult to teach the American people the doctrine of reasonable military preparedness. War correspondents who have witnessed maneuvers in Panama have correctly pointed out the inadequacy of the canal defenses, particularly in their air aspect. The necessity for fortifying another site can hardly be expected to correct existing defects nor to simplify the problems of defense.

A word about commercial prospects: That some impetus to local business will result in the area contiguous to the Nicaragua Canal, if built, is certain. The field of development of Central American trade and industry may seem large, but is generally over-rated. Its possibilities certainly do not, in themselves, warrant any considerable expenditures. The Nicaragua route opens no new important markets or sources of raw material that are not equally served or reached by the Panama route.

International Factors.—The final element now deals with political conditions. The stability of the situation in Panama, as a virtual protectorate of the United States, is attested to by its relative tranquillity. The United States has exclusive perpetual canal rights in Panama. Although the United States also has exclusive perpetual rights for the construction of the Nicaragua Canal, obtained by treaty with Nicaragua in 1916 for a consideration of \$3 000 000, the conditions for the Nicaragua route are not so favorable. The San Juan River which is the essential route for the canal between the Atlantic and Lake Nicaragua is along the international boundary between Costa Rica and Nicaragua. The establishment of a Canal Zone as a buffer State under United States jurisdiction would probably be welcomed or opposed by Central American strategists, depending on their motives. By some the construction of a canal *via* Nicaragua may be hailed as a panacea to cure all Central American ailments, but it is doubtful whether it would enjoy a very care-free existence in the troubled times of to-day.

Conclusion

The study of the predictable future growth of Isthmian canal traffic has indicated that the peak-load capacity of the Panama Canal will probably not be reached until 1970. The current discussions, in Congress and the press, therefore, appear to be a quarter of a century in advance of necessity. Not that foresight is ever to be condemned, but extensive preparations for a situation which may be materially altered by unforeseeable developments seems unjustifiable. Perhaps events to transpire in the future will place a different complexion on both the necessity and the advisability of a Nicaragua Canal. Neither appear to exist at present. In short, the problem belongs to posterity.

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PAPERS AND DISCUSSIONS

CHARACTERISTICS OF CENTRIFUGAL OIL PUMP

By Michael D. Aisenstein,* Esq.

Synopsis

Recently, a movement has been growing to use centrifugal pumps on oil pipe lines and at refineries. Because they are quantity pumps and have such merits as low first cost, simplicity of operation, steady flow, and adaptability for direct connection to high-speed machinery, centrifugal pumps attract all industries which have to deal with pumping machinery. Whereas the efficiency of a properly designed centrifugal water pump runs very high, this efficiency decreases when the same pump is used in a viscous fluid.

It is evident that, when pumping oil (besides all ordinary mechanical and hydraulic losses, for a viscous liquid), a new loss enters which dominates all the others. This is the viscosity or the internal resistance loss. The effect of viscosity is to decrease the head capacity and to increase the disk friction. This lowers the efficiency of the centrifugal pump correspondingly.

It is important to know the laws which govern the decrease in the head capacity and the increase of horse-power which affect the efficiency of the centrifugal pump. It is also important to know the limiting value of viscosity for each type and size of centrifugal pump at which it is practical to pump viscous oils. Moreover, as it is a comparatively easy matter to test a pump on water, it is of great advantage to have a method by which it is possible to predetermine what a centrifugal pump of known water charateristics will do when pumping a viscous oil.

By characteristics is meant the curves which show the relation between the capacity, head, efficiency, and power at a constant speed. These curves are usually obtained from an actual test using water as pumping medium.

The original ideas which will be presented in this paper occurred to the writer while engaged in research work at the Hydraulic Laboratory of the University of California. These ideas, supplemented by numerous investi-

Note.—Discussion on this paper will be closed in **December, 1928.**

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gations made in the Test Laboratory of the Byron Jackson Pump Company, form the basis for this paper.

Notation

The following notation has been adopted:

d_2 = the diameter at outlet of impeller, in inches.
b_2 = the breadth between the impeller disks or shrouds at the diameter, d_2, in inches.
z = number of vanes.
A = the free exit area at the impeller periphery, in square inches = $(\pi d_2 - z t) b_2$.
D = the equivalent diameter of the free area at the impeller periphery $= \sqrt{\frac{4A}{\pi}}$, in inches.
ν_0 = kinematic viscosity of oil in poises.
ν_w = kinematic viscosity of water in poises = 0.01.
s = specific gravity.
t = vane length at periphery (outlet).
G_0 = U. S. gallons per minute when pumping oil.
G_w = U. S. gallons per minute when pumping water.
H_0 = head, in feet, when pumping oil.
H_w = head, in feet, when pumping water.
BHP_0 = brake-horse-power when pumping oil.
BHP_w = brake-horse-power when pumping water.
HP_f = horse-power absorbed to overcome the disk friction.
x = an exponent.
HP_m = horse-power absorbed to overcome journal and stuffing-box friction = $BHP_w (1 - e_m)$.
B = distance from the center of a disk to the wall of the casing.
e_m = mechanical efficiency.
T = torque, in foot-pounds.
e = pump efficiency.
U = peripheral velocity of the impeller, in feet per second.
g = gravitational constant = 32.2.
θ = angle.
ω = angular velocity.
n = an exponent.
r = radius.

General Considerations

The suitability of a pump for a given service is determined by the characteristic curves. A given pump operating at a constant speed can deliver a certain quantity of liquid at a given head. As the head is reduced the

capacity increases. The maximum value of head occurs near the point of the zero discharge. Theoretically, the maximum head which can be produced by an impeller is,

$$H = \left[\frac{\text{Diameter, in inches} \times \text{revolutions per minute}}{1\,840}\right]^2$$

Actually, the maximum head may be more or less than the calculated value, depending mainly on the design of the impeller and of the casing. From given characteristics, new curves can be computed for speeds above and below the operating speed for the same pump. This is possible because, if the diameter of the impeller is kept constant:

1.—The capacity varies as the first power of the speed;
2.—The head varies as the second power of the speed; and,
3.—The power varies as the third power of the speed.

It must be emphasized that this is correct if the variation in speed is not very great. Every pump is designed for a certain speed and it will not operate with the same efficiency at a speed very different from that for which it is designed.

Furthermore if the impeller speed is kept constant, provided the width of the impeller is constant:

(*a*) The capacity varies approximately as the diameter;
(*b*) The head varies as the square of the diameter; and
(*c*) The horse-power varies as the cube of the diameter.

These relations are true approximately. Actually, the head drops below the recalculated value. As a rule the efficiency of a pump falls off with the cut, although in some instances when the runner is too large for the casing the efficiency may improve for a relatively small cut.

These laws do not hold near points at which the pump cuts off; that is, points that mark sudden drops of head, efficiency, and horse-power. These drops occur when the suction inlet of the impeller is relatively small and the rate of discharge becomes too great for the net opening area of the impeller and when the pump becomes choked. No matter how wide open the valve on the discharge side of the pump is, the capacity stays constant.

Derivation of the Head-Capacity Equations

The effect of pumping different liquids with the same pump, operating at a constant speed, may be stated as follows:

1.—The capacity will vary inversely as some power of the kinematic viscosity of the liquid:

$$\frac{G_0}{G_w} = \frac{1}{\left(\frac{\nu_0}{\nu_w}\right)^n} \quad \text{(1)}$$

2.—The head will vary inversely as the square of the same power of kinematic viscosity of the liquid:

$$\frac{H_0}{H_w} = \frac{1}{\left(\frac{\nu_0}{\nu_w}\right)^{2n}} \quad \text{(2)}$$

Hence, the problem is to determine the various factors which affect the exponent, n.

The writer has found that n varies directly with the quantity, G, to some power, a, and the kinematic viscosity, ν, to some power, b. It varies inversely with the free exit area, A, to some power, c. Hence,

$$n = \frac{G_w{}^a \ \nu_0{}^b}{A^c} K \quad \text{.......................} (3)$$

Substituting the values:

$$A = (\pi \ d_2 - z \ t) \ b_2 = \frac{\pi \ D^2}{4} \quad \text{....................} (4)$$

and,

$$K \left(\frac{4}{\pi}\right)^c = K' \quad \text{..........................} (5)$$

In Equation (3), the coefficient may then be written,

$$n = \frac{G_w{}^a \ \nu_0{}^b}{D^{2c}} K' \quad \text{.........................} (6)$$

This value of n may then be substituted in Equations (1) and (2), in order to obtain values of G_0 and H_0.

From experiments, the numerical values of the exponents were found to be:

$$K' = \frac{1}{15}; \ b = 0.25; \text{ and } 2 \ c = 6 \text{ for } b_2 > \tfrac{1}{2} \text{ in.}$$

For small values of b_2 the exponent, n, is very sensitive to any variation in the width of the impeller and it is always better to obtain this dimension by actual measurement. The value of $2 \ c$ for $b_2 < \frac{1}{2}$ in. appeared to vary from 6 to 7. For viscosities of 1 600 S.s.u. (Saybolt seconds universal), and higher, $a = 1.5$. For viscosities less than this, a is determined from an empirical equation,

$$a = (\mu \times 0.1055)^{0.08} \quad \text{......................} (7)$$

in which, μ is viscosity, in Saybolt seconds universal.

The net result is that Equation (6) becomes,

$$n = \frac{G_w{}^a \ \nu_0{}^{0.25}}{15 \ D^6} \quad \text{..........................} (8)$$

Analyzing Equations (1) and (2), it may be seen that $\dfrac{1}{\left(\dfrac{\nu_0}{\nu_w}\right)^n}$ is ordinarily less than unity, because $\dfrac{\nu_0}{\nu_w}$ for viscous oils is usually greater than 1.

In case $\dfrac{\nu_0}{\nu_w} = 1$, the term, $\left(\dfrac{\nu_0}{\nu_w}\right)^n$, becomes equal to unity, $G_w = G_0$, and $H_w = H_0$; that is, when liquids have the same kinematic viscosity, the head capacity curves are the same. On the other hand, the term, $\left(\dfrac{\nu_0}{\nu_w}\right)^n$, may become equal to unity when the exponent, n, is equal to zero. This is the case when $G_w = 0$; that is, when there is no flow, the "static head" at the shut-off valve is inde-

pendent of viscosity and is approximately equal to $\frac{U^2}{2\,g}$. Since U is the peripheral speed of the impeller the "static head" depends on the speed of the prime mover and the dimensions of the impeller and is practically independent of the viscosity.

By studying the different factors that influence the exponent, n, it is possible to improve the design of a centrifugal oil pump. It may be easily noticed that the effect of viscosity on the head-capacity curve decreases rapidly with increases in the size of the pump.

Derivation of the Horse-Power Equations

Consider, next, the horse-power required by a centrifugal oil pump. In general, the horse-power taken by a centrifugal water pump is composed of (1) horse-power to overcome the disk friction; (2) hydraulic horse-power; and, (3) horse-power necessary to overcome the friction in the stuffing-boxes and bearings which, for a uniform speed, is practically constant.

The first two items increase with the viscosity of the liquid, and the brake horse-power of a pump operating in oil, may be expressed in terms of brake horse-power of a water pump by the following formula:

$$B\,H\,P_0 = s\,H\,P\,f_w\left(\frac{\nu_0}{\nu_w}\right)^x + (B\,H\,P_w - H\,P\,f_w - H\,P_m)\left(\frac{\nu_0}{\nu_w}\right)^n s + H\,P_m \quad \text{(9}$$

The factor, $H\,P\,f_w$, is the horse-power absorbed in overcoming the disk friction in water and is equal to the difference between $H\,P\,f_w$ corresponding to the outlet diameter of the impeller and the $H\,P\,f_w$ corresponding to the inlet diameter of the impeller.

Consider the first term of Equation (9); that is, the disk friction. When an impeller is revolving in a case of a centrifugal pump filled with any liquid, it acts as a brake. (See Fig. 1.) If U is the velocity of the element at the

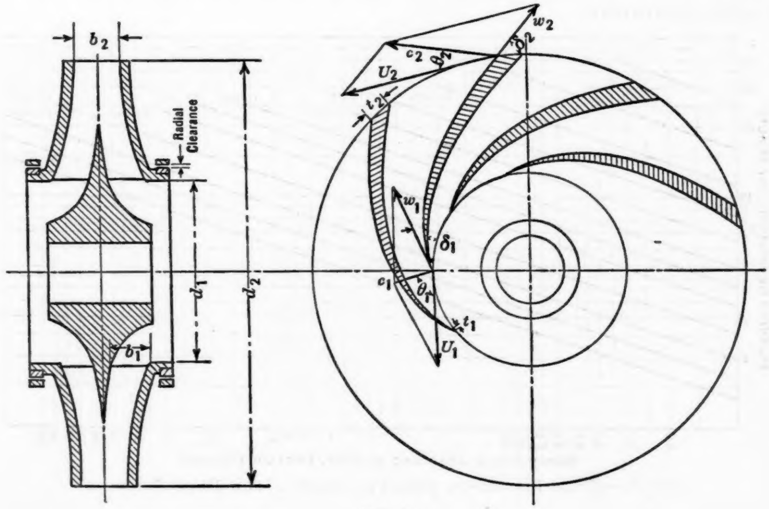

FIG. 1.

radius, r, and f is a coefficient of friction, the frictional resistance, R, equals

$$R = f\ U^n\ A \quad \text{.......................... (10)}$$

or the frictional resistance on an element of the area, $d\ A$,

$$d\ R = f\ U^n\ d\ A = f\ r^n\ \omega^n\ (r\ d\ \theta\ d\ r) \quad \text{................ (11)}$$

The torque necessary to rotate the disk in liquid is,

$$d\ T = r\ d\ R = f\ r^{n+2}\ \omega^n\ d\ \theta\ d\ r \quad \text{.................. (12)}$$

$$T = \frac{4\ \pi\ f\ \omega^n}{n+3}\ r^{n+3}\ \text{ft-lb} \quad \text{..................... (13)}$$

Taking into account the thickness, b,

$$T' = 2\ \pi\ r\ b\ f\ r\ r^n\ \omega^n = 2\ \pi\ b\ f\ r^{n+2}\ \omega^n \quad \text{............. (14)}$$

$$T'' = T + T' = 2\ \pi\ f\ \omega^n\ r^{n+2} \left[\frac{2\ r}{n+3} + b\right] \text{ft-lb.}$$

and,

$$H\ P\ f = \frac{T''\ \omega}{550} = \frac{2\ \pi}{550}\ f\ \omega^{n+1}\ r^{n+2} \left[\frac{2\ r}{n+3} + b\right] \quad \text{......... (15)}$$

The coefficient, f, is influenced by the ratio of $\frac{B}{2\ r}$, by the roughness of the surface, the viscosity, and the density of the liquid. Many empirical equations have been published which are a modification of Equation (15), namely, those of Unwin, Lashe, Banki, Gibson and Ryan, Le Conte, V. Zur Nedden, etc.

Nearly all of them show that the horse-power absorbed to overcome the disk friction varies approximately as the fifth power of the diameter and as the third power of the speed. The writer uses Professor Banki's formula:

$$\text{Horse-power} = \frac{0.0000000961\ N^3\ d^5}{550}$$

in which, N = revolutions per minute, and d = diameter, in feet. This is graphically represented in Fig. 2, which makes it possible to determine the $H\ P\ f_w$ direct for a given speed and diameter of the impeller in a pump with the usual clearances.

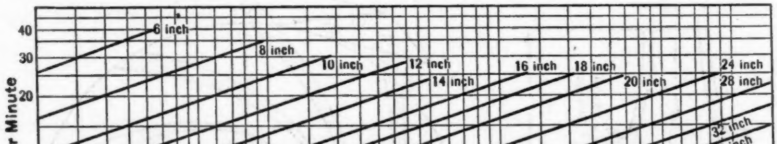

FIG. 2.—FROM PROFESSOR BANKI'S FORMULA FOR DISK FRICTION.

If an impeller is revolved at a constant speed in two different liquids, say, first in water and next in a viscous oil, then the relation between horse-power, kinematic viscosity, and specific gravity, is,

$$\frac{H\,P\,f_0}{H\,P\,f_w} = \left(\frac{\nu_0}{\nu_w}\right)^x \frac{s_0}{s_w} = \left(\frac{\nu_0}{\nu_w}\right)^x s_0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

The exponent, x, is a function of viscosity and is determined for a given viscosity from Fig. 3. This curve was plotted by the writer from experi-

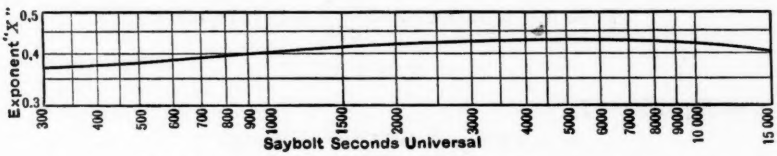

FIG. 3.—VALUES OF x TO BE USED IN EQUATION (9).

mental data. The second term of Equation (9) is the hydraulic horse-power. It is evident that, due to internal friction, it takes a greater amount of power to move a more viscous fluid and this can be expressed as:

$$H\,H\,P_0 = (B\,H\,P_w - H\,P\,f_w - H\,P_m)\left(\frac{\nu_0}{\nu_w}\right)^n \quad \ldots\ldots\ldots\ldots (17)$$

in which, $(B\,H\,P_w - H\,P f_w - H\,P_m)$ is the hydraulic horse-power when pumping water. The exponent, n, was found to be the same as in Equation (6).

When designing a centrifugal pump it is always important to see that the percentage of horse-power absorbed in order to overcome the disk friction be a reasonable percentage of the total horse-power consumed by the pump. This is especially important when choosing an oil centrifugal pump, because the disk friction constitutes a tremendous loss in such a case, depending on the viscosity of the liquid. The problem of selection, the proper number of stages, dimensions, and angles of an impeller, must be studied very carefully.

In general, for a centrifugal water pump not more than 5% of the total horse-power is allowed for the disk friction. When the required head is high a single-stage pump would require either that the diameter of the runner be large or that the pump be run at a high speed. For this reason a multi-stage pump, with several runners of relatively small diameter, operating in series, is used.

In viscous liquids, the disk friction may be many times greater than in water, and it is always necessary to determine in advance whether a pump which is good for water service is suitable for oil and then select a proper motor.

Illustrative Example.—Consider a 5-in. single-stage pump operating at 1750 rev. per min. Let $d_2 = 10\frac{1}{2}$ in.; $b_2 = \frac{3}{4}$ in.; $z = 8$; and, $t = 0.31$ in.

From Equation (4),

$$A = (\pi\, 10\tfrac{1}{2} - 8 \times 0.31)\,\tfrac{3}{4} = 22.9 \text{ sq. in.}$$

$$D^2 = 22.9\,\frac{4}{\pi} = 29.2$$

and,

$$15\ D^6 = 373\ 000$$

Next, predetermine the characteristics for the pump for oil with a viscosity of 675 S.s.u. and a specific gravity of 0.924. Then the kinematic viscosity in poises is computed by the empirical equation,

$$\nu_0 = \mu \times 0.00216 - \frac{1.8}{\mu} \qquad (18)$$

$$\nu_0 = 675 \times 0.00216 - \frac{1.8}{675} = 1.457 \text{ poises}$$

$$\nu_0^{0.25} = 1.1$$

From Equation (7),

$$a = (675 \times 0.1055)^{0.08} = 1.405$$

The computations for the head capacity are shown in Table 1.

TABLE 1.

G_w.	H_w.	$G_w^{1.405}$.	n.	$\left(\frac{\nu_0}{\nu_w}\right)^n$.	$\left(\frac{\nu_0}{\nu_w}\right)^{2n}$.	G_0.	H_0.
880	45.0	18 500	0.0399	1.22	1.49	780	29.5
800	62.5	11 800	0.0349	1.19	1.42	673	44.0
600	85.0	7 760	0.0229	1.13	1.28	531	66.4
500	92.0	6 310	0.0186	1.10	1.21	454	76.0
400	97.0	4 570	0.0135	1.07	1.14	374	85.0
300	100.0	3 020	0.00891	1.06	1.12	283	89.3
200	100.8	1 699	0.00501	1.03	1.06	194	95.0
100	100.1	646	0.00191	1.01	1.02	99	98.0
0	99.0	0	0	1.00	1.00	0	99.0

Next, refigure the horse-power for the same viscosity and specific gravity. When a pump is lifting water, and is operating at full load,

$$B\,H\,P_w = 18.3$$

Assuming the mechanical efficiency of the pump to be 97%,

$$H\,P_m = B\,H\,P_w\,(1 - 0.97) = 0.55 \text{ h.p.}$$

Next, from Fig. 2, the horse-power absorbed to overcome the disk friction in water for an impeller, 10½ in. in diameter, revolving at 1 750 rev. per min., is:

$$H\,P\,f_w = 0.5$$

The $H\,P\,f_w$ for the inlet diameter of the impeller is negligible ($d_1 = 5$ in.).

From Fig. 3 the exponent, x, corresponding to a viscosity of 675 S.s.u., is:

$$x = 0.39$$

$$H\,P\,f_0 = 0.924 \times 0.5 \times 145.7^{0.39} = 3.23$$

Then tabulate the computations for $B\,H\,P_0$ in the same manner as for the head capacity. (See Table 2.)

TABLE 2.

G_0.	$B\,H\,P_w$.	$G_0{}^{1.405}$.	n.	$\left(\frac{\nu_0}{\nu_w}\right)^n$.	$s\,(B\,H\,P_w - H\,P f_w - H\,P_m)\left(\frac{\nu_0}{\nu_w}\right)^n$.	$B\,H\,P_0$.
730	20.0	10 500	0.031	1.17	20.5	24.3
673	19.5	9 500	0.028	1.15	19.3	23.1
531	17.5	67 500	0.020	1.105	16.2	20.0
454	16.3	5 400	0.016	1.083	15.0	18.8
374	14.7	4 100	0.0121	1.062	13.2	17.0
283	13.0	2 800	0.0083	1.042	11.5	15.3
194	11.0	1 130	0.0033	1.017	9.5	13.3
99	9.0	630	0.0019	.....	7.7	11.5

Plotting the given values of H_0, $B\,H\,P_0$, and efficiency, against G_0 as abscissa, the oil-pumping characteristics of a centrifugal pump are found as based on water performance. Similar curves were plotted for viscosities of 1 350 and 3 400 S.s.u. (See Figs. 4, 5, and 6.)

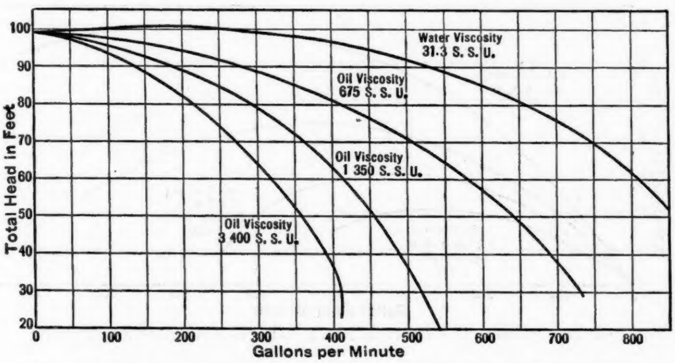

FIG. 4.—HEAD CAPACITY CURVES FOR A 5-INCH SINGLE-STAGE PUMP.

PROCEDURE IN COMPUTING THE OIL CHARACTERISTICS OF A MULTI-STAGE PUMP

The case of a multi-stage pump is more complicated than that of a single-stage pump. Oil passing through the first stage, on account of losses, is heated; and the more stages it has to pass, the more pronounced is the heating effect. The heat given to the oil raises its temperature. The change in temperature changes the viscosity, and the drop in head in the first stage will be greater than in the second stage; in the second stage greater than in the third, etc. Therefore, it is necessary in the case of a multi-stage pump, to calculate the losses in the first stage and express them in British thermal units; determine the increase in temperature; look up the new viscosity corresponding to this temperature; and compute the head-capacity curve for the second stage, basing the calculations on the new viscosity which evidently is less than that for the first stage. After the head-capacity and brake-horsepower curves are obtained for each stage, it is necessary to add them to obtain characteristics of the whole pump.

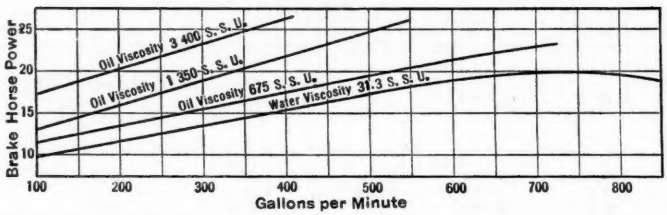

FIG. 5.—BRAKE-HORSE-POWER CURVES FOR A 5-INCH SINGLE-STAGE PUMP.

This is a very long and tedious task and for ordinary practice it is sufficiently accurate to calculate the head capacity, brake horse-power, and efficiency for a given viscosity for one point, usually at the operating point, in the same manner as has been done for a single-stage pump. This will be the first approximation. The next step is to take the difference between the horse-power input in the pump and the horse-power output of the pump, con-

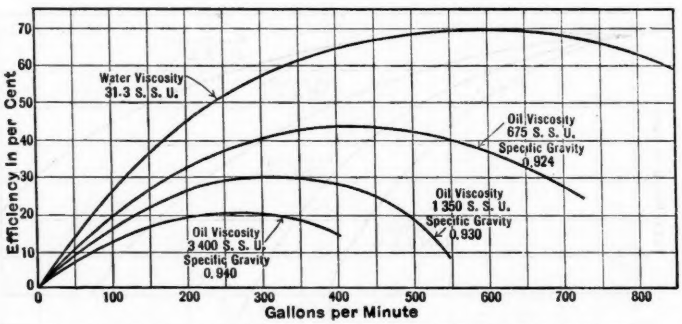

FIG. 6.—EFFICIENCY CURVES FOR A 5-INCH SINGLE-STAGE PUMP.

vert this difference into British thermal units, and knowing the specific heat of the oil (which varies from 0.45 to 0.505), determine the rise in temperature and corresponding viscosity. With the new viscosity determined, the head-capacity, brake horse-power, and efficiency curves for the entire range may be calculated.

Illustrative Example.—Consider a four-stage pump operating at 1 750 rev. per min., which has the following dimensions: $d_2 = 15$ in.; $b_2 = \frac{1}{2}$ in.; $t = 0$ in. (vanes filed to a point); and $z = 6$.

Then, from Equation (4),

$$A = \pi \times 15 \times \frac{1}{2} = 23.5$$

$$D^2 = 30$$

and,

$$15\ D^6 = 405\ 000$$

Let the temperature of the oil to be pumped be 104° Fahr. The viscosity corresponding to this temperature equals 1 000 S.s.u. (See Fig. 7.) Then,

$$\nu_0 = 1\,000 \times 0.00216 - \frac{1.8}{1\,000} = 2.16$$

$$\nu_0^{0.25} = 1.21$$

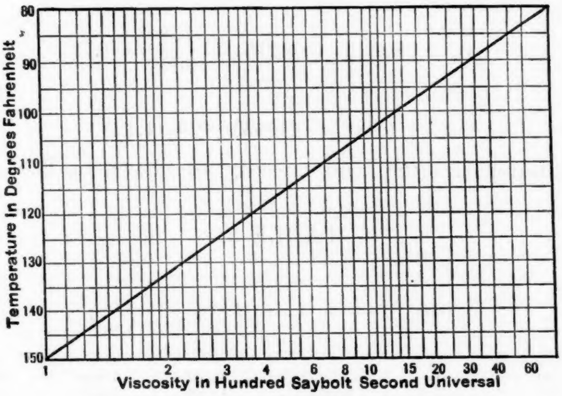

FIG. 7.—VISCOSITY-TEMPERATURE CURVE.

From Equation (7),

$$a = (1\,000 \times 0.1055)^{0.08} = 1.452$$

For the first approximation: $G_w = 600$; $H_w = 720$; $G_w^{1.452} = 10\,800$; $n = 0.0387$; $\left(\frac{\nu_0}{\nu_w}\right)^n = 1.23$; $\left(\frac{\nu_0}{\nu_w}\right)^{2n} = 1.51$; $H_0 = 476$; $G_0 = 487$; $G_0^{1.452} = 8\,000$; $n = 0.0239$; $\left(\frac{\nu_0}{\nu_w}\right)^n = 1.137$; and $BHP_w = 162$.

Assuming 97% mechanical efficiency of the pump and 0.95 specific gravity of the oil,

$$HP_m = 162\,(1 - 0.97) = 4.87 = \text{a constant}$$

HPf_w for an impeller 15 in. in diameter, revolving at 1 750 rev. per min., is 2.75 (from Fig. 2). Therefore, for four stages (neglecting HPf_w for the inlet diameter),

$$HPf_w = 4 \times 2.75 = 11$$

and the disk friction horse-power, when pumping oil, is found by Equation (16),

$$HPf_0 = 0.95 \times 11 \times 216^{0.402} = 90.2$$

and,

$$BHP_0 = 4.87 + 90.2 + (162 - 11 - 4.87)\,0.95 \times 1.137 = 252.7$$

The horse-power lost in heat is equal to,

$$BHP_0 - \frac{G_0\,H_0\,s}{3\,960} = 252.7 - \frac{487 \times 476 \times 0.95}{3\,960} = 197.2$$

The rise in temperature, in degrees Fahrenheit, is,

$$t_2 - t_1 = \frac{\text{Horse-power lost in heat} \times 42.44}{\text{Specific heat} \times G_0 \times \text{weight}}$$

In this case,

$$t_2 - t_1 = \frac{197.2 \times 42.44}{0.5 \times 487 \times 8.33 \times 0.95} = 4.3^\circ \text{ Fahr.}$$

Hence, the new temperature is 108.3° Fahr., and the corresponding viscosity (from Fig. 7) is 800 S.s.u. Using this viscosity as the basis for the second approximation,

$$\nu_0 = 800 \times 0.00216 - \frac{1.8}{800} = 1.723$$

$$\nu_0^{0.25} = 1.145$$

$$a = (800 \times 0.1055)^{0.08} = 1.425$$

The computations for the head capacity are shown in Table 3.

TABLE 3.

G_w.	H_w.	$G_w^{1.425}$.	n.	$\left(\frac{\nu_0}{\nu_w}\right)^n$.	$\left(\frac{\nu_0}{\nu_w}\right)^{2n}$.	G_0.	H_0.
0	830	0	0	1.000	1.000	0	830
200	900	1 900	0.00536	1.028	1.057	195	850
400	845	5 100	0.0144	1.077	1.160	372	728
600	720	9 000	0.0254	1.140	1.300	527	552
800	515	10 350	0.0292	1.160	1.350	690	382

From Fig. 3, the exponent, x, corresponding to viscosity of 800 S.s.u. is 0.390, therefore,

$$H\,P\,f_0 = s \times H\,Pf_w \left(\frac{\nu_0}{\nu_w}\right)^x = 0.95 \times 11 \times 172.3^{0.390} = 77.8$$

The computations for $B\,H\,P_0$ are given in Table 4.

All values from Tables 3 and 4 are plotted in Fig. 8, and the characteristics of the pump for oil are obtained.

TABLE 4.

G_0.	$G_0^{1.425}$.	n.	$\left(\frac{\nu_0}{\nu_w}\right)^n$.	BHP_w.	$H\,Pf_w$.	$H\,Pf_0$.	$H\,P_m$.	$s\,(B\,H\,P_w - HPf_w - HP_m)\left(\frac{\nu_0}{\nu_w}\right)^n$.	$B\,H\,P_0$.
0	0	0	1.000	...	11	77.8	4.87	.....	.....
195	1 850	0.0052	1.027	110	11	77.8	4.87	97.5	180.2
372	4 500	0.0127	1.670	140	11	77.8	4.87	125.8	208.5
527	7 500	0.0212	1.105	170	11	77.8	4.87	161.9	244.6
690	11 000	0.0310	1.175	190	11	77.8	4.87	194.0	276.7

It must be mentioned that the point of cut-off and the lower part of the head-capacity curve, when calculated by the derived equations, may give values which are less than the actual. The reason for this is that the efficiency of the pump decreases toward the cut-off point and at this point practically the whole power is consumed in overcoming losses.

This means that there is a greater heating effect and, consequently, a greater decrease in viscosity at points closer to the cut-off point, and it is evident that the actual head-capacity curve may deviate from the theoretical head capacity which is based on constant viscosity.

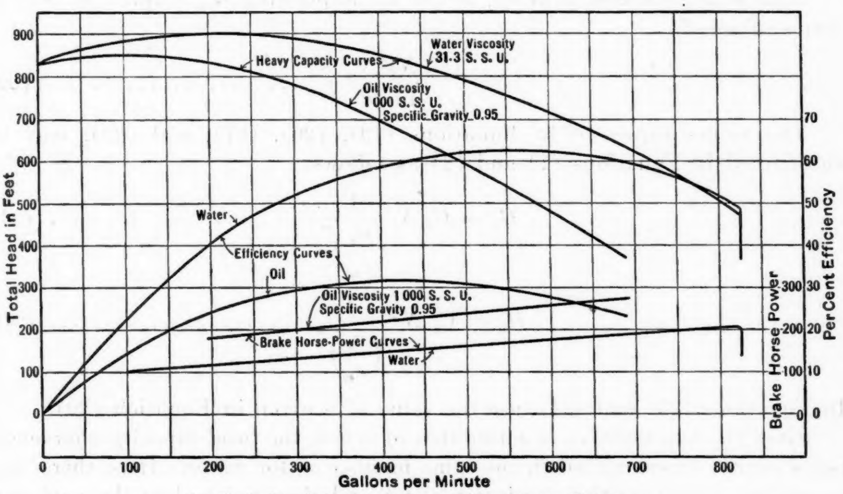

FIG. 8.—3-INCH, 4-STAGE MULTIPLE PUMP.

Another point which must be emphasized is the effect of leakage. It is well known that the leakage past the runner wear-rings and the casing wear-rings of a centrifugal pump when lifting water is a relatively great loss, particularly for high heads and small pumps. It is evident that the leakage decreases with the viscosity, resulting in an increase of the volumetric efficiency of the pump. This probably is the reason for the increase of the exponent, c, for pumps of small sizes.

CALCULATION OF THE OIL CHARACTERISTICS FOR DIFFERENT SPEEDS

Suppose a pump had been tested at two speeds with oil and the performance from oil to oil was refigured for each speed. How do these oil performances for two different speeds compare? In other words, is it correct to refigure from a given performance of a pump on oil at one speed to another speed in the same way as is done for water?

Since in case of water the head varies as the square of the speed and the capacity as the first power of speed,

$$H_w = C_1 N^2 \qquad (19)$$

$$G_w = C_2 N \qquad (20)$$

and,

$$H_w = C G_w^2 \qquad (21)$$

In Equations (19), (20), and (21), N is the speed, in revolutions per minute, and C is a constant, for a given point, and is determinable for a given pump from the head-capacity curve at a given speed. Hence, if the speed is doubled,

the capacity is also doubled, and the head is increased four times. In other words, when the pump is operated in water the reduction of a point at different speeds follows a line of relatively constant efficiency. This is a straight line with a slope of 2:1 when plotted on logarithmic paper.

This is not the case with oil, however. Expressing G_w (Equation (8)) in terms of speed,

$$n = \frac{(C_2 N)^a \nu_w^{0.25}}{15 D^6} \quad \text{(22)}$$

The values expressed in Equations (19), (20), (21), and (22), may be substituted in Equations (1) and (2) as follows:

$$G_0 = C_2 N \frac{1}{\left(\frac{\nu_0}{\nu_w}\right)^n} \quad \text{(23)}$$

$$H_0 = C_1 N^2 \frac{1}{\left(\frac{\nu_0}{\nu_w}\right)^n} \quad \text{(24)}$$

In Equations (23) and (24), use the value of n given in Equation (22).

Since the exponent, n, is a function of speed, the head-capacity characteristics cannot be calculated in the same manner as for water. It is, therefore, necessary to refigure the speeds for water and then recalculate the performance from water to oil as shown. The same result will be obtained if the constants, C_1 and C_2, are determined from Equations (19) and (20) and the values of these constants and the speed, N, are substituted in Equations (23) and (24).

Illustrative Example.—Consider the 5-in. double-suction, single-stage pump which was used in Example 1 and calculate the characteristics for 1 450 rev. per min.

From Table 1,

$$G_w = 800; H_w = 62.5 \text{ at } 1\,750 \text{ rev. per min.}$$

From Equations (19) and (20),

$$C_2 = \frac{800}{1\,750} = 0.457$$

$$C_1 = \frac{62.5}{1\,750^2} = 0.0000205$$

By Equation (22),

$$n = \frac{(0.457 \times 1\,450)^{1.405} \times 1.1}{373\,000} = 0.02714$$

Substituting these values in Equations (23) and (24) and assuming that $N = 1\,450$ rev. per min.,

$$G_0 = 0.457 \times 1\,450 \frac{1}{(145.7)^{0.02714}}$$

$$G_0 = 577$$

$$H_0 = 0.0000205 \times 1\,450^2 \; \frac{1}{(145.7)^{2 \times 0.02714}}$$

$$H_0 = 32.7$$

In like fashion all the points can be determined and a smooth head-capacity curve can be passed through these points.

As an illustration of the effect of the viscosity on pumps of larger size, the performance of a 10-in. double-suction pump is plotted in Fig. 9 for water and for oil with a viscosity of 1 200 S.s.u.

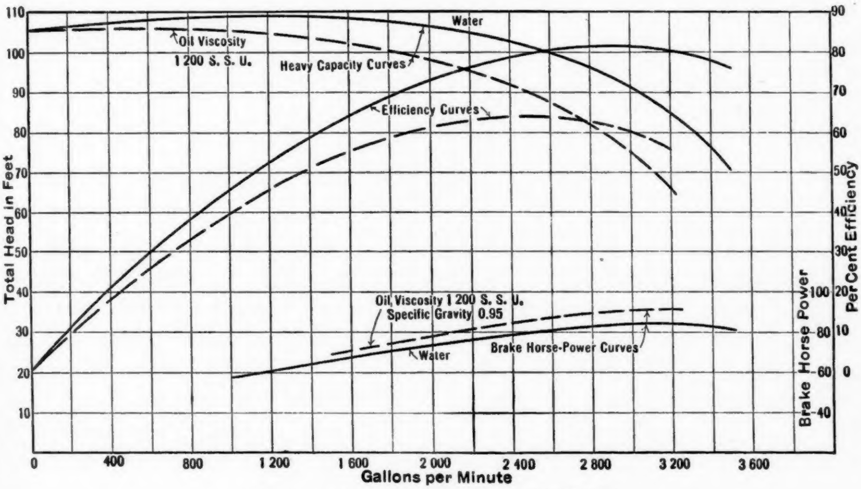

FIG. 9.—10-INCH DOUBLE-SUCTION PUMP.

CONCLUSIONS

From these considerations, the following conclusions may be made:

1.—It is possible to predetermine, with a sufficient degree of accuracy, the performance of a centrifugal oil pump by observing its characteristics as obtained on water.

2.—The effect of the viscosity is to decrease the head capacity and increase the power.

3.—When the pump is working in viscous oil, the increase in horse-power is influenced mainly by the increase in the disk friction due to viscosity.

4.—The head-capacity characteristic and the efficiency of a centrifugal pump as applied to viscous oils improve with the increase of the size of the pump.

5.—The volumetric efficiency, particularly that of a low specific speed centrifugal pump, improves with the increase of the viscosity.

6.—In a multi-stage pump the performance is improved because of the heating of the oil in the pump by losses. This is more noticeable in pumps of low specific speed.

7.—It seems that an impeller with "flat" characteristics will be more efficient when pumping viscous oils than an impeller with "steep" characteristics.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

STREET DESIGNING FOR VARIOUS USES*

By George William Tillson,† M. Am. Soc. C. E.

What is a street? The dictionaries define it generally as a public highway, paved or unpaved, in a city, town, or village. Unfortunately, engineers have held no different view. Some years ago the writer stated that highway engineers were more backward in their practice than any other branch of the profession. This was disputed in the press, but the intensive research work that has been done during the last few years on roads and streets clearly demonstrates its truth.

Street Functions.—In attempting to discuss the matter in its widest sense, it will be necessary to consider all the functions of a street. Broadly, they are:

(1) To facilitate traffic, both vehicular and pedestrian, as much as possible.

(2) To furnish light and air to all buildings that may be erected along it.

(3) To furnish opportunity for the construction of public utilities of all kinds.

(4) To add by its dimensions and treatment to the beauty of the municipality.

Traffic.—It must be admitted that the original purpose of a street was somewhat as given in the dictionaries, but as cities grew in size and their wants increased the more comprehensive needs arose and demanded attention. In order to lay out streets to comply with the first principle, however, a full knowledge of what the traffic will be, both as to quantity and character, is essential. Clearly, a residential street must differ from one devoted to business or heavy traffic, not only in width of roadway, but in kind of pavement. The acceptance of this principle requires that it be known in advance whether the community is to be as a whole for residential or business use.

Note.—Discussion on this paper will be closed in December, 1928.

* Presented at the meeting of the City Planning Division, Denver, Colo., July 14, 1927.

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With the rapid and changing developments of American cities, due to the birth of new activities such as the automobile, the forecast becomes exceedingly difficult.

Light and Air.—It is not strange that the original street planners did not consider the provision for light and air as necessary. The buildings were neither high nor close together, so that they formed no obstruction. When, however, the streets had been built up solidly and the structures began actually to soar toward the skies, the problem of ventilation and lighting became important.

Accommodation of Public Utilities.—In the present civilization when each household requires so many aids from outside to carry on its daily tasks, provision must be made for furnishing them. Even within the writer's memory it was thought that when sewer, gas, and water mains had been provided, the sub-surface use of the street was satisfied; but now, in addition, there are ducts, or pipes, for electric currents, telephone wires, steam for heating, and, in some cities, for wires or cables for street-car propulsion, and even subways for the cars themselves. All these needs came so gradually that at first the pipes were located in a haphazard manner with no attempt to keep a record of their position. In the larger cities they soon formed a regular network and it became a serious problem to find room for any new installations. This was particularly true at intersections, but even within the blocks it became necessary to dig test pits to find what part of the sub-surface was unoccupied. In New York, N. Y., and Philadelphia, Pa., systematic surveys have been made to locate the underground construction. However, space is required not only below the surface, but also on the surface and in the air.

Where street cars are propelled by overhead trolleys provision must be made for supporting the necessary wires, an important point, as one of the cardinal principles in street administration, is to minimize obstructions. If, however, it is generally recognized that street cars on the surface are an evil, it is also true that in most cases they are a necessary evil. In some European cities the cross-wires supporting the trolley wires are attached to buildings; the general practice is to attach them to poles erected either on the curb or in the center of the street. In the Borough of Manhattan, New York City, no poles are allowed, all wires—telephone, electric, or trolley—being laid underground.

Beautification of the Municipality.—It is only quite recently that this function has been considered of much importance, since city planning has received so much, and such intensive, study. Probably one of the first boulevards, as they are now called, was built in the then City of Brooklyn, N. Y., from Prospect Park to Coney Island, a distance of 5½ miles, with only one change of direction. It is 210 ft. wide, with a center drive of 70 ft. and a 30-ft. park, a 25-ft. roadway for heavy traffic, and a 15-ft. sidewalk space on each side.

Perhaps the best instance of beauty and utility is the boulevard system of Chicago, Ill., which undoubtedly is the finest in the world. The topography as well as the vision of the early planners have made this possible without a prohibitive expense.

Original Plans.—Few cities were laid out originally with any definite idea and still fewer with any definite purpose. Capital cities present the best opportunity for real city planning as their requirements can be pretty clearly foreseen. St. Petersburg (now Leningrad), Washington, D. C., and Canberra, Australia, are good examples of this. Undoubtedly, they are the only ones of any size.

Peter the Great literally built St. Petersburg, beginning on May 22, 1703. The location was boggy and marshy. During some of his previous travels he had spent nine months working in the shipyards of Zaandam, a thriving town in Holland, near Amsterdam. While there he became acquainted with the general plan of that city which was founded in the early part of the Thirteenth Century on land very similar to that of the proposed City of St. Petersburg. He, therefore, adopted that plan and brought architects from Amsterdam to lay out his city. The historian recounts that as there were no wheelbarrows the earth for filling was brought in baskets, bags, and even in the skirts of the workers' clothes. Also, that the laborers were brought from the surrounding countries under his control by the thousands, that 30 000 buildings were erected during the first year, the first three being a fort, a church, and a hotel. He adds incidentally that owing to the unhealthy conditions 100 000 of these laborers died in that period. This was the first large capital city of modern times, laid out as such.

When it was decided to found a capital city for the United States, General Washington secured the services of Major L'Enfant, a French Army Engineer. His plan, adopted in 1791, was most acceptable. While it has been changed somewhat in minor details, as a rule most of the modifications have been restored, and the original plan carried out almost in its entirety. In the writer's opinion it is destined to be the finest capital city in the world.

A possible exception is Canberra, the new capital of Australia. Competitive designs for this city were asked by the Government and the one submitted by Mr. Walter B. Griffen, of Chicago, was accepted. Work has proceeded to such a stage that the temporary Parliament Building was opened by the Duke of York on May 9, 1927.

These three cities are undoubtedly the only ones laid out and constructed as capital cities for nations. Salt Lake City, Utah, built in what was practically a desert, is a State Capital, but it was neither designed nor constructed as such. Brigham Young, however, had the vision of the city planner. His idea was:

"* * * to divide the city into blocks of ten acres with eight lots in a block of one and a quarter acres each. The streets to be wide. No house will be permitted to be built on the corners of the streets, neither petty shops. Each house will have to be built so many feet back from the street and all the houses parallel with each other. The fronts are to be beautified with fruit trees".

European countries as a rule have beautiful capitals, but they have been developments rather than original conceptions. In France, Paris, for example, has a wonderful boulevard system, but, although the city existed as a small capital in 292 A. D., it remained for Baron Haussmann, who was what would

now be known as Minister of Public Works, to lay out the boulevards, the one bearing his name having been completed in the winter of 1926-27. In England, London, as usually thought of, is made up of twenty-nine different municipalities, so it is not strange that its street system is not regular and logical. Vienna, Austria, is another capital where the original layout (inside the Ringstrasse) is exceedingly complicated. This can be duly appreciated only by a walking trip through it.

In America, the older cities simply started and then "just grew". Lower Manhattan (New York) had originally no plan. Very few of its streets extend even from river to river. Pearl Street, for instance, begins at State Street near the Battery, runs easterly, then northerly, and, finally, westerly, terminating at Park Row, opposite City Hall. In 1807 a Commission was formed to lay out a plan for the Island of Manhattan north of this lower section. This Commission reported in 1811, recommending the present system of streets as far north as 155th Street.

Boston, Mass., has probably the worst street system of any of the large American cities. The smallness of its original area mitigates the difficulties somewhat. Western cities, however, were as a rule mapped before they were built up to any great extent. Chicago has the gridiron system; many of the streets running east and west extend miles into the suburbs. There are very few that extend diagonally.

The original plat of Omaha, Nebr., provided for streets 100 ft. wide, with 20-ft. alleys both running east and west. The blocks were 264 by 284 ft., including the alleys. When the city outgrew this plat, no attempts were made to reproduce it, but the streets in the adjacent additions were laid out according to the ideas of the property owners. This practice, however, was eventually curbed.

Denver, Colo., was planned on the gridiron system, but the streets run northeast and southwest, the idea being that sometime during the day the sun would be accessible to every side of a detached house. As in the case of Omaha, subsequent developments ignored this idea.

Macon, Ga., has streets 180 ft. wide running in one direction and 150 ft. at right angles. The writer knows of no city in this country other than Salt Lake City, that has streets of these widths.

These examples show what has been accomplished in some instances by real forethought, but usually with none at all. In no case does there seem to have been an attempt to provide for particular needs except perhaps in the Manhattan Plan of 1811, where the streets running from the North to the East River were laid out every 200 ft., with the idea that provision should be made for river-to-river traffic, railroads not having been thought of at that time. Even so, the widths were only 60 ft., the exceptions being 14th, 23d, 34th Streets, etc., which were 100 ft. wide and about $\frac{1}{2}$ mile apart, giving 20 blocks to the mile.

For any engineering design the important considerations are: What will be its functions and for what will it be used? Very few streets have been planned with these ideas in mind. To give the designer proper information

it would be necessary to know what was to be the character of the proposed community. Was it to be general, business and residential; suburban, with only enough business to care for the general needs; or one strictly for business, the residences being only for the wants of the employees. Towns of this latter type were constructed during the World War.

Street Systems.—Three street systems are in general use: (*a*) the so-called gridiron system, the streets being at right angles to each other; (*b*) the gridiron system with diagonal streets superimposed upon it; and (*c*) the curvo-linear system, used mainly in residential districts. All these systems are often modified somewhat according to special conditions.

It should be remembered that the gridiron system is the most economical, both as to street space and the use of the property, as square lots can be utilized to better advantage than those of any other shape. Such a system, however, presents a monotonous appearance, and access to different parts of the city is not as easy as with the superimposed system. The latter makes many triangular shaped lots and large street intersections, but it provides places for small parks, monuments, and other ornamental structures, so that beautifying the city becomes much easier than by any other plan.

The curvo-linear plan is used to best advantage in residential towns that are irregular in their topography. It gives, as a rule, large lots, but permits spacious lawns that can be readily landscaped, and its streets require no excessive grading, as that can be avoided by their proper location. It is particularly adapted to a well-to-do community, but where land is not expensive it can be used to good advantage for workmen's homes. Without special adornment it gives an attractive effect. Stuttgart, Germany, is a good example of the curvo-linear system.

It is not easy to ascertain the probable use of the street, but any plan should be flexible to some extent, so that if occasion requires it can be modified without great expense. Cities that are expected to grow considerably often have features that will give some idea of the probable location of the business district, such as, for instance, a water-front on a river, or a large body of water, or the location of railroad tracks. When no such indication exists, the planner must use his best judgment and bolster up his decision by appropriate zoning.

Widths.—Besides the location, the entire width of the street must be determined and also the width of the roadway.

In deciding on the total width the first consideration is light and air. With low buildings, this is not a vexing question, but, with structures rising to such enormous heights as are now being built in large cities, it is most important. It is generally admitted that it is not practicable to solve the problem entirely by increasing the street width. Instead, this width should be made what the general traffic requires, and then the desired result should be obtained in the structures themselves by setting back the outside walls certain distances above certain heights (according to the width of the street) as the height increases.

This width should then be considered from a traffic standpoint, forecasting the probable increase from year to year. That this is extremely difficult has been very forcibly demonstrated by the experience of the past few years with the enormous increase in the use of automobiles.

Consideration must also be given to future public service requirements. It is not enough to say that corporations should look out for themselves. They are absolutely necessary, and the ease with which they can build their structures will be reflected in the cost of furnishing their product and in their rates.

If the city is to be so large as to require surface street cars, it must be determined whether the wires are to be above or below ground. If above, the location of poles must be fixed, and if below, the required conduit, in almost every case, will be logically in the center of the street probably directly over the sewer. Similarly, it is important to decide upon the location of each utility, whether city or privately owned, and to take care when any is constructed that it is placed in its own location, not only horizontally, but vertically. In the case of sewers, the street grades often necessitate a change from the general plan.

The Chicago Regional Planning Association has accomplished great results in this way. By careful and diplomatic work it has obtained the consent of the ruling authorities to agree to one general location for all the probable utilities in nearly all the different towns and villages within its district.

Roadway Widths.—As a general proposition streets should be wider in warm, than in cold, climates. Provision should be made for streets of different traffic, that is, heavy trucks, passenger automobiles, and street cars. If these uses are determined in advance, improvements will follow known conditions. The total width, however, must be divided for vehicular and pedestrian traffic.

Three possibilities govern this division, that is, whether the street is to be used for wholesale or retail business, or for strictly residential purposes. In many cities, wholesalers receive their goods directly from trucks across the sidewalk. Consequently, the sidewalk space should be no greater than necessary. On a street used by wholesalers entirely there is comparatively little pedestrian traffic, so that it is logical to make the roadway wide and the sidewalk narrow. The location of alleys, too, will mitigate this evil, making it possible to handle freight in the rear of the buildings. The original plat of Memphis, Tenn., has alleys running in both directions across the blocks. The writer believes that the time is not far distant when goods will be received and delivered within the warehouses, either in spaces especially provided for them on the first floor, or in the basement by means of tunnels. This system is in use to some extent now, and greatly relieves congestion on the street proper.

On retail streets the problem is not so simple. As a rule, surface cars occupy the center space of the roadway, thus greatly reducing its capacity, and pedestrian traffic reaches its maximum. The character of the buildings, both as to size and use, and the frequency of the passing cars, will be the factors

to be considered in the determination of width. This division, however, is not necessarily permanent, as the curbs can be easily moved at comparatively little expense when changing conditions require.

A conspicuous example of this is Fifth Avenue, New York City. The total width is 100 ft. Originally, the roadway width was 40 ft., leaving a sidewalk space on each side of 30 ft., but the property owners were allowed to use the 15 ft. adjacent to their lots for areas and stairways to their first floors. The congestion caused by automobiles became so great that in 1908 the Board of Estimate and Apportionment passed an ordinance making the roadway 55 ft. wide and ordering the householders to remove the area fences and steps, permitting an encroachment of 2½ ft. only. This gave space for six lines of automobiles instead of four, as formerly, and a sidewalk of 20 ft., where previously it had been but 15 ft. in many places.

For residential streets the traffic conditions need not be studied so carefully; in fact, some people think residential streets should not be planned for any through travel. It must be remembered, however, that in this age small villages often find themselves on the direct route between large cities, and, although in such cases the through traffic should be by-passed wherever possible, a considerable amount of it will traverse the town. In residential streets a careful distinction must be made between sidewalks and sidewalk space, as this space need not be entirely occupied. To what extent it is so occupied depends greatly on its width and the size of the municipality.

In laying out a residential town where property is cheap, liberal street widths should be used, as the roadway space which is the expensive part to maintain can be made as narrow as necessary, leaving the area between the curb and street line to be treated as desired. Some engineers wish the walk to be located next the curb, while others prefer it to be adjacent to the property. The first plan gives a larger lawn in front of the buildings, but the latter makes it a simpler matter to widen the roadway should it ever become necessary.

In a town site designed for the use of the operatives of any large corporation the writer believes the streets should be comparatively narrow, planned for small cottages, so that there may be a house for each family at a very small expense, instead of larger buildings occupied by several families. Also, for a town of this character, the roadway width need not be greater than 24 ft. in most cases. The appearance of a wider street can be created by setting all buildings back a given distance from the street line. The set-back, however, reduces the size of the back yard, so it is a matter for determination whether this extra width shall be in front of, or behind, the house.

Admitting that business streets should be wider than residential ones, it remains to be determined what and where they should be. The writer believes that, in home communities, the tendency is to allot too much area to business purposes. Where no factories exist the function of retailers is to supply the wants of the citizens. To have too many dealers is detrimental, and, while business property always commands a higher price than residential property, too large an area set aside for this purpose injures a town, from stand-

points of both economy and beauty. These remarks will also apply to the outlying districts of large cities.

In determining roadway widths provision should be made for a specific number of traffic lanes. For roads, where higher speeds are used than in streets, engineers are largely agreed that a 10-ft. width per lane is needed, but the writer thinks a lesser width can be used in most city streets. An expedient that has been used in several districts, where the general traffic is toward town in the morning and away from it in the afternoon, is to divide the roadway into lanes designated by white strips, the number of lanes for use being determined by the direction of the greater traffic. A three-lane street would always have two lanes for the more important use, according to the need. Another expedient for a narrow roadway is to provide for one-way traffic. This has proved very satisfactory.

Business Requirements.—In suburban communities one would expect to find the area of retail business districts to vary as the distance from the large city varies. To ascertain the relationship, the Chicago Regional Planning Association made a survey of nineteen cities and villages in the Chicago Region, ranging in population from 3 500 to 12 000. The unit of comparison was the ratio of retail business frontage actually in use to the population served. This varied from 36.8 to 64.8 ft. per 100 population. Fifteen of the towns were classed as residential, and four as industrial, communities.

Kenosha, Wis., an industrial city of 52 700 people, 52 miles from Chicago, had a ratio of 53.9 ft., while Evanston, Ill., a residential city although adjacent to Chicago, had a ratio of 63.1 ft. per 100 population. The mean ratio of the nineteen communities was 47.6 ft.

Taking four villages on the Chicago, Burlington, and Quincy Railroad, at distances of 11, 17, 21, and 28 miles from Chicago, the ratios were 40.8 46.8, 49.8, and 64.8 ft. respectively. A study of such figures gives an approximate idea of what ratio should be used in determining the business frontage in laying out a residential community.

Ornamental Streets.—As America developed, both in size and wealth, the desire to beautify its cities rapidly increased. This was accomplished by the creation of parks and by making certain streets and locations particularly attractive. Streets are beautified in several ways by erecting notable buildings with pleasing grounds as well as monuments along the sides, by a special treatment of the street surface itself, and also by artistic lighting.

Historians state that the ancient City of Memphis was connected with the Pyramids by a broad roadway, 2 leagues long, having a paved and well kept driveway, lined on both sides with temples, mausoleums, porticoes, monuments, statues, etc., in fact, the modern boulevard. The Ringstrasse (Vienna) that took the place of the wall around the old city, has its sides adorned by statues and monuments. The Sieges-Allee in Berlin leading to the statue of Bismarck, has marble groups at regular intervals along its sides.

As recognized in recent times, however, the boulevard consists of a broad right of way with special treatment, such as the finances of the community will permit. One thing city planners insist upon—that these streets shall

have a vista, something to gaze at and rest the eyes rather than a distant and indefinite view. When the boulevard is comparatively short, this is not so difficult. It is better if an objective can be had at each end.

Pennsylvania Avenue, Washington, was intended to run from the Capitol to the White House, but the Treasury Building has been constructed across it, shutting off this view. Unter den Linden, Berlin, running westerly from the River Spree, has the Brandenburg Gates at its western terminus.

The Champs de Elysees, Paris, a magnificent street, runs from the Tuileries to the Place de l'Etoile, from which radiates fourteen streets, and at the center of which is the celebrated Arc de Triomphe.

As might be expected Europe has many magnificent and beautiful boulevards, as it has had money and talented architects for many years. America is beginning to have both money and planners and the desire to use them so that with these and the ability to perform work so much more rapidly than a hundred years ago, in a comparatively short time its cities will have works of which to be proud.

Super-Highways.—The most recent development in roads is the so-called "super-highway". It is the direct result of an established need and want, an outstanding example of highway construction for a known use. Had any one, a few years ago, even proposed a road 200 ft. wide, he would have been considered insane. Yet at Detroit, the standard width of the super-highway has been made 204 ft. and the entire system comprises 217 miles (31 miles within the city limits) connecting cities 25 to 30 miles distant.

Chicago Plans.—Around Chicago, other comprehensive plans have been adopted. The Chicago Regional Planning Association is working on a proposed Three-State Boulevard (Figs. 1 and 2) 200 ft. wide, extending from

FIG. 1.

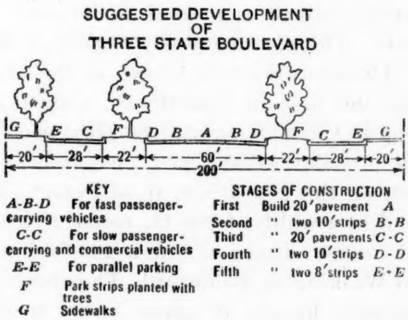


FIG. 2.

Milwaukee, Wis., to Michigan City and La Porte, Ind., the line dividing at Chesterton, Ind. The total distance is 183.5 miles, and it is planned to have the pavement free from all railroad grade crossings. The route as tentatively

selected by-passes Chicago, thereby relieving that city of a large amount of through traffic. No plans have been made for immediate construction, the idea being to have the lines adopted, so that the right of way can be obtained before real estate prices become prohibitive. Property owners along the route will also be enabled to develop their land with a view to the coming improvement. This road will lie in three States and nine counties. The ease with which the right of way is being obtained by voluntary dedication by the property owners is remarkable.

In Wisconsin, some of the right of way is being dedicated without cost to the State or county. The State Highway Department may lay the pavement where needed, because part of the route follows on existing State trunk highway. In Illinois, promises of right of way have been given for 61 out of 82 miles. It is hoped that certain sections will coincide with the State Bond Issue System and the remainder may be paved unit after unit by State aid. In Indiana, more than half the distance is promised, and part is already dedicated. The pavement may be laid by the counties under the Three-Mile Road Law, and if the right of way is furnished it may be accepted by the State as a State Road.

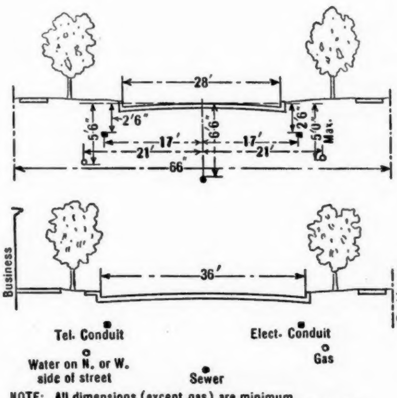
Du Page County Super-Highways.—Supplementing the Three-State Boulevard are three highways, 200 ft. wide, established by the Supervisors of Du Page County, extending from the Kane County line across Du Page County to the west line of Cook County, connecting with leading thoroughfares into Chicago. All these highways would cross the Three-State Boulevard, giving direct connections north and south.

The routes of the center and south roads have been definitely determined and proceedings for the rights of way are being actively conducted, the property owners in nearly all cases being willing to cede their rights, the routes being through undivided land.

The development of these highways is being furthered by the Metropolitan Super-Highway Association. This Association, organized to carry out this system, offered a prize for the best development of the surface of these roads. The plan adopted was that of Mr. R. E. Toms, of Montgomery, Ala.

The central route, known as the Butterfield Road Super-Highway, is perhaps the first in importance, connecting as it does with the heart of the Loop in Chicago, *via* 22d St., Roosevelt Road, and Washington Boulevard. It will probably be the first one under construction. Commencing at Mooseheart, on the Fox River, it will extend eastward along the present Butterfield Road in a width of 200 ft., connecting at the Du Page-Cook County line with 22d Street. There will be a 100-ft. highway connecting with Roosevelt Road and Washington Boulevard. Thus centrally located, it will adequately drain and efficiently handle all motor traffic from the roads leading into it from both the north and south. It is not expected, of course, that this plan will be carried out at once. Two one-way pavements, each 40 ft. wide, will be built eventually, but the idea is first to construct the outer half of each roadway, the remainder being completed when traffic requires and funds are available.

Possible arrangements of streets to accommodate subsurface structures and various conditions of traffic are illustrated in Figs. 3 and 4. The locations given in Fig. 3 are suggested by the Chicago Regional Planning Association, having been approved by its Committee on Drainage, Sanitation, and Water Supply.



RECOMMENDED LOCATION OF UNDERGROUND UTILITIES

FIG. 3.

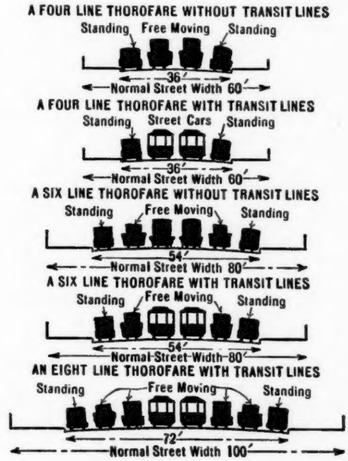


FIG. 4.

There has also been proposed by the Advisory Committee of the Cook County Forest Preserve:

"the creation of a great forest preserve highway, 600 ft. wide, connecting various tracts and makings all parts accessible by automobile. The highway would be more than a mere automobile road. Striking the present tracts, it would consist of motor driveways, parking places, equestrian paths, and walks, all shaded by dense foliage. It would require an addition of 4 000 to 5 000 acres to the preserve to carry out the program."

To what extent this plan will be carried out is uncertain at present, but it shows that the general public is interested in the beautification of the modern city as well as its material progress. Too much credit cannot be given to all these planners for their forethought in providing routes to be developed and completed as future needs demand. By so doing, the enormous expense that cities and counties have been put to, in order to provide rights of way for necessary transportation routes, will be avoided. Their efforts will be appreciated and applauded by future generations.

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THE RELATION BETWEEN EARTHQUAKES AND ENGINEERING SUBSTRUCTURES*

By Henry D. Dewell,† M. Am. Soc. C. E.

The Problem Stated

The limits of this paper will allow but a rather superficial treatment of the subject, but sufficient to outline the problem, and suggest phases for discussion. Much is yet to be learned concerning the effect of earthquakes upon engineering structures; much, about earthquakes themselves; and much, about the stresses set up in engineering structures due to earthquakes. For this ignorance, engineers as a class, are not without blame. In general, they have not been enough interested to give the subject the intense study and thought required to design intelligently against an earthquake. In the past the subject of the relation of earthquakes to engineering structures has been one of intermittent interest to engineers, the time interval being the periods between shocks of a disastrous intensity.

It is the geologist and the seismologist who have studied both the earthquake and the effect of the earthquake on engineering structures. The seismologist is not, primarily, an engineer, and frankly admits his lack of practical and even theoretical knowledge of the stresses and strains in engineering structures. Yet, because of the lack of continued interest by the engineer, it is the seismologist who has set forth the conclusions generally accepted as representing the relation of earthquake to engineering structures.

Kinds of Earthquakes

In order to anticipate the effect of an earthquake upon engineering structures, it is necessary to understand the cause of the earthquake and its effect upon the soil in which the engineering substructure is located. In fact, if knowledge of this subject were complete; if the nature and limitations of the forces and deformations of the soil were known; the remainder of the

Note.—Discussion on this paper will be closed in December, 1928.
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problem—how to plan the substructure to resist these forces and deformations—would be comparatively simple. Hence, a review of what is believed to be known about earthquakes is in order. In what immediately follows, no attempt is made to mention specifically what is fact; what is hypothesis, based on the evidence available; and what is conjecture.

Earthquakes are divided into two classes: (1) Volcanic and (2) tectonic; in other words, earthquakes associated with volcanoes and earthquakes resulting from slips along fault planes. Such fault planes divide the earth's crust into "blocks"; hence, the aptness of the adjective with the Greek root denoting "carpentry". Probably 90% of the earthquakes of the present era, and certainly all those in California, have been of the tectonic type.

Earthquake Waves

Certain causes, concerning which one can do little more than conjecture, build up stresses between one block of the earth's crust and an adjoining block. When these stresses have reached the limit of resistance of shear between the two blocks, a slip occurs. The effect of such slip is to send out through the earth's crust elastic waves—somewhat as ripples are sent out along the surface of a pool when a pebble is thrown into the water.

When an isotropic body (having the same elastic properties in all directions about a point) is subjected to an internal disturbance, two principal types of waves are sent out from the source. In the first type, the particles vibrate along lines normal to the wave front, that is, parallel to the direction of propagation. In the second type, the particles vibrate in planes parallel to the wave front, or perpendicular to the direction of propagation. During the passage of the waves of the first type, the material of the solid is alternately compressed and rarified. The waves causing these phenomena are known as "longitudinal", "condensational", or "compressional" waves. The second type waves are known as "transverse" or "distortional" waves. Note that, during the passage of these waves, the particles may vibrate in any direction in a plane perpendicular to the line of propagation.

Now, when these elastic waves pass from one medium with a certain density to another medium of another density, both the longitudinal and transverse waves are modified. Each may be resolved into four new waves, a reflected longitudinal, a refracted longitudinal, a reflected transverse, and a refracted transverse. The number and types of such subsidiary waves depend partly on the angle of incidence of the earthquake wave, and partly on the relative values of the elastic moduli of the two media.

Taking into consideration the wide variation in the densities and elastic qualities of different parts of the earth's crust, it must be concluded that the earthquake waves which reach any point on the earth's surface are complex in nature. In the area on the surface, immediately above the seat of disturbance, called the "epicenter", it is also evident that greater complexity must occur, and that waves of both types with various modifications of each type will be found. Seismographic records verify this conclusion.

Phases of Earthquake Disturbances

At a point some distance from the surface of the earthquake, there is a more definite arrangement and sequence of elastic waves. The record of the seismograph will show three phases. The first phase is a small but sudden displacement from the straight line, followed by rapid irregular vibrations, each lasting 1 sec., or more. In this phase, there are occasionally sudden reinforcements of the motion. These vibrations are termed the "first preliminary waves" or "primary waves".

The second phase is usually marked by a considerable increase in amplitude. The first vibration is probably succeeded by irregular movements, with an amplitude which may increase to a maximum and then decrease. The waves of this phase are known as the "second preliminary tremors" or "secondary waves".

The first and second phases comprise the elastic waves which have reached the point of record by a route through the earth. The waves of the first phase are principally of a compressional longitudinal type; the waves of the second phase are largely of the distortional type.

The third phase comprises the waves which have traveled along the surface, and are of the distortional or transverse type. They are of a longer period, and the movements, are, in general, larger and more regular than in the preceding planes. These waves are known as the "long waves" or "large waves". The first few undulations are followed by a series of regular harmonic vibrations, some of which probably attain the largest amplitude of the whole motion. After these large movements, the undulations become less regular, and the movement gradually dies down.

Points on the earth's surface in the epicentral zone may feel the compressional waves coming directly from the source, their vertical components being much larger than the horizontal. In general, however, the vertical components of the elastic waves are relatively unimportant. The engineer needs be concerned principally with the horizontal components.

Effects on Substructures

The subject of elastic waves due to an earthquake is very complex. To summarize present knowledge of the characteristic disturbances of these elastic waves on engineering substructures:

(1) Any structure situated immediately over an earthquake fault on which an actual surface slip takes place must be ruptured. The force causing this rupture will be equal to the resistance offered, whatever the amount.

(2) Fault planes subsidiary to the main fault plane may be affected and become local sources of intensified action.

(3) The intensity of earthquake shocks diminishes rapidly with increasing distance from the fault of origin. A shock may cause damage to a structure situated many miles from the active fault plane.

(4) The intensity of the disturbance manifested depends on the nature of the soil, the damaging effect being greater in loose or alluvial soil, than in rock. The wave length, amplitude, period, and maximum effective accelera-

tion are greater in alluvial soils than in rock. In the latter, the elastic waves are of small amplitude and great frequency—more like a shiver than the slower undulating motion that occurs in the marshy ground.

(5) In general, an earthquake begins with tremors of which there may be 5 or more per sec. In the principal part of the disturbance, the average period of vibration ranges from 0.5 sec. to 1.5 sec. The third part of the disturbance is a decrease in intensity of the vibrations.

(6) The horizontal component of the acceleration of the earthquake wave is usually the most destructive. The vertical component, in general, can be ignored, although there have been exceptions in which the vertical acceleration was important. The location of a structure with reference to the epicenter of the shock is the principal factor which determines the importance of the vertical movement.

(7) Accelerations ranging from 0.003 g to 1.00 g (g being the acceleration due to gravity) have been observed or estimated in earthquakes. An intensity of 0.4 g, or more than 13 ft. per sec. per sec., is not uncommon in great shocks, but this, as well as higher intensities, is of such local occurrence that, from the standpoint of general structural design, it need not be considered. In the California earthquake of 1906, the estimated acceleration of the shock in the vicinity of Third and Market Streets, San Francisco, in hard sand, 7 miles from the fault of origin, was estimated at from 0.6 ft. to 2.6 ft., or from 0.02 to 0.08 that of gravity. In the same earthquake, the estimated acceleration in "made" and marsh land ranged as high as 12 ft. per sec. per sec.

(8) The force of the earthquake on the structure is dependent on the mass of the structure and acceleration of the elastic wave. The horizontal force is the product of the mass of the structure (which moves with the same acceleration as the ground, as measured in amount and direction) by the horizontal acceleration of the ground. Expressed mathematically, in pounds and feet:

$$F = M\,a = \frac{w}{g}\,a$$

in which,

F = horizontal force, in pounds.
M = mass.
a = acceleration of ground, in feet per second per second.
g = acceleration of gravity, in feet per second per second.

(9) In loose, dry, ground the initial tremors may be damped to some extent, but the soil is liable to shake down in the following vibrations. In loose, water-filled ground, the initial shock is transmitted with full force on account of the rigidity of the water and the subsequent shaking of large amplitude, because the material offers little elastic resistance to distortion. If a foundation rests on material of high elastic resistance, the effect of the shock is very much smaller, and a structure built on firm rock is not likely to suffer damage unless it is very weak or very near the fault of origin.

(10) The intensity of an earthquake shock is greatest at the surface of the ground. Sir John Milne found a marked reduction in the intensity in a pit

10 ft. deep. There has been found to be a perceptible difference between the intensities at the surface and down in mines.

Mr. Charles Davison writes that:

"In the zone of greatest disturbance in the Riviera earthquake of 1887, the shock was very weak, or not felt at all in the tunnels of the Nice to Genoa Railway, and no one of these tunnels was damaged in the slightest degree. There is also a marked difference in the strength of a shock in slight hollows or excavations at the surface and on adjoining ground. For instance, in the central tract of the Mino-Owari earthquake of 1891, the railway lines were everywhere more or less disturbed except in small cuttings. Even if the cuttings were not more than 20 or 50 feet in depth, the rails and sleepers were unmoved.

"Experiments by Sekiya and Omori at Tokio in the years 1887-1889, using two similar seismographs, one at the bottom of an 18 ft. pit, and the other on the surface within a few yards from the pit, were compared for thirty earthquakes, three severe and the rest slight. For the latter, it was found that the average amplitude, maximum velocity and maximum acceleration differed but little on the surface and in the pit, each being slightly greater on the surface. It was in the ripples of these earthquakes that the greatest difference was manifest, the amplitude being about twice, the maximum velocity three times, and the minimum acceleration five times as great at the surface as in the pit. Owing to their much shorter period, the ripples at the surface have a maximum acceleration from five to ten times as great as that of the large undulations. Thus, it would seem from these observations that the ripples are in great part smoothed away in the pit, and that there should be much less destructive action in houses with foundations rising from deep pits than in those built on the surface."

(11) Secondary effects of earthquakes are (*a*) avalanches, usually of rock and earth; (*b*) earth slumps; (*c*) earth lurches; and (*d*) earth fissures. Avalanches and earth slumps are confined to more or less steeply sloping ground. Earth lurches and earth fissures occur in level ground, as well as on slopes. In soft, sandy, or marshy ground, surface waves due to the earthquake are often seen. These waves may have an average length of 30 ft., and a height of 1 ft.

Importance of Location.—From the preceding discussion, it is evident that the location of engineering structures is all important. Any structure on ground that suffers actual differential displacement must be injured, for it must suffer the same relative displacement as the soil.

Thus, any structure on a fault line on which an actual slip occurs must be ruptured. Any structure in soft marshy ground will, under the action of an earthquake, probably suffer great damage. Deep pile foundations, if extending through the soft ground to firm material underneath, will help very materially, and if the foundations are well tied together, the structure will probably be safe against destruction.

Application to Structures

It remains to review briefly the probable action of usual types of engineering structures.

Dams.—The requirements of reservoir capacity very often determine the position of dams over faults. The engineer in locating such a structure, in

seismically active regions, should have the geological conditions carefully studied. Earth-fill dams behaved remarkably well in the California earthquake of 1906. The great San Andreas fault passed directly through the San Andreas Dam on the San Francisco Peninsula. This was an earth-fill dam, 800 ft. long and 93 ft. high, and, although ruptured, no water escaped, and no permanent damage resulted. The Pilarcitos Dam was 1¾ miles from the same fault line. It also is an earth-fill dam, 95 ft. high. It was not damaged. The Upper Crystal Spring Dam—another earth-fill dam—75 ft. high, was crossed by the fault line. Due to a change in the original operating plans, water stood on both sides of the dam. The dam was ruptured, but apparently not seriously damaged. Two other earth-fill dams in the Santa Cruz Mountains were on the fault line. Both were cracked. One suffered no leakage, the other lost its impounded water by reason of shattered outlet pipes. Three high earth-fill dams were situated on the east side of San Francisco Bay. One had a concrete facing. Although these were severely shaken, no appreciable damage resulted. The Crystal Springs Dam, a concrete structure of the gravity type, 146 ft. high, was a few hundred feet away from the fault line on which a surface slip of from 6 to 7 ft. took place. The longitudinal axis of the dam was parallel to the fault. No damage resulted. Had the axis been perpendicular to the fault line, the result might have been disastrous.

In the Santa Barbara earthquake of 1925, the Sheffield Dam, situated on one of the fault planes believed to have been active, failed. The dam, however, is said on good authority to have had a known weakness.

In general, it would seem that as against earthquake damages, earth-fill dams are comparatively safe. In the order of type, with respect to safety against earthquake, it would seem reasonable to place the concrete gravity type second. The writer knows of no cases of rock-fill or multiple-arch dams having been subjected to heavy earthquakes. The possible rupture of the concrete faces of the rock-fill dam, and the weakness of the arch against distortion, leaves no doubt as to the weakness of the two types for resisting earthquakes.

Pipe Lines.—The question of earthquake damage in the case of pipe lines is even more a question of location than in the case of dams. Pipe lines crossing a fault slip will be ruptured. The great San Francisco fire of 1906 which followed the earthquake, was the result of broken pipe lines. Pipe lines on soft ground in which actual displacements occur, will be ruptured irrespective of the material of which they may have been constructed. The use of trusses to carry such pipe lines over marshy ground, will not save the line, as was proven in the California shock of 1906.

Sewers.—The same comments made on pipe lines apply to sewers, particularly to lines of vitrified pipe. Reinforced concrete pipe should fare better.

Tunnels.—Observations on tunnels, lead to the conclusion that, in general, they are affected only by the displacement or loosening of the material in the sides and roof, caused by the shaking ground. Careful geological investigation of the site, and proper provision for the conditions found, will give safety. Unless actually crossing an active fault, tunnels in rock would seem to be

reasonably safe from damage, even though close to the active fault. Tunnels in earth, near an active fault, may be made fairly safe by proper timbering.

Bridge Piers and Abutments.—This class of engineering substructures has fared badly in many cases in heavy earthquakes. It is probable that no bridge pier was ever designed to resist an earthquake in the sense that a definite acceleration of the ground was considered.

Foundations.—The value of deep foundations for protection against an earthquake seems quite definitely proved, although the quantitative factor is not known. Could the structure be kept free from actual contact with the upper, or surface, ground, except by slip-joints, the evidence available points to the conclusion that much of the energy of the earthquake would never reach the superstructure. The relation of the fault line to the superstructure, as determining whether the structure lies within the probable epicenter, is perhaps an important factor in the case.

Consideration of the nature of the disturbances of the soil during the passage of the elastic seismic waves, points to the value of a unit foundation, as a reinforced concrete mat, under a building. When this is economically impossible, heavy ties, as of reinforced concrete, between the individual footings, are believed to be of great benefit. In the California earthquake of 1906, well-designed foundations of various types were reported to have withstood the shock, irrespective of type. The evidence in this case was largely negative, as few foundations could be observed.

Personal observation of some relatively heavy unreinforced concrete building foundations in Santa Barbara, which were sheared by the earthquake, emphasizes the great danger of "cold" construction joints, as planes of weakness against earthquakes. The shock of the earthquake is transmitted to the building through the foundations. The force is measured in terms of the mass of the superstructure, not that of the foundation alone. The swaying of the superstructure resulting from the application of this oscillating force may reverse the bending due to the normal vertical loads, in portions of the foundation. The writer has observed shattered footings that apparently had been almost pulverized by rocking of the superstructure. The plentiful use of steel reinforcement in foundation piers and walls is of great importance in strengthening the foundation so that it may safely resist forces of the earthquake, and thus protect the superstructure. In all these cases a "lean" concrete mixture is not economy.

Conclusion

The proper design of engineering substructures for resistance to earthquake, must consider: (1) The location with respect to known seismically active faults, or faults likely to become active; (2) the nature of the soil in which the structure is to be constructed; (3) the nature and type of the superstructure; (4) the assumption of a probable acceleration of the ground during an earthquake; and (5) the design of the substructure for the assumed acceleration with a reasonable factor of safety. Unity of substructure, as far as is reasonably warranted from the standpoint of cost, is a desideratum.

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BASE MAPS FOR REGIONAL PLANNING*

By Harland Bartholomew,† M. Am. Soc. C. E.

Just as American cities have developed without comprehensive plans, regions are now developing in much the same loose, disjointed, and uneconomical manner. A "region" is a super-city produced by the modern forms of rapid transit and the automobile. It may be merely an enlarged city with a common commercial and industrial center; or it may be a group of cities that have grown into a large metropolitan community with common physical or economic conditions. A region may be a very large area and not a metropolis, as, for instance, the Ruhr Valley in Germany, but for purposes of this paper it will be assumed that a region is one large community developed around a large city, the most familiar type.

To deal scientifically with problems to be met in regional planning, first it is necessary to have intimate knowledge of the physical characteristics of the region, since these will control the character and extent of growth and the nature of plans to be adopted. The basic purpose of regional planning is the adaptation of an area to its best form of human occupancy—providing for the logical and economical opportunities to develop commerce and industry and to distribute population in a manner producing the most agreeable living conditions with ample opportunities for the enjoyment of life. City planning has not yet accomplished these objectives to any marked degree, for most city growth took place in advance of the time when the need for comprehensive planning was fully understood and appreciated. It is hoped that the recurrence of wasteful mistakes experienced in city growth will be prevented by comprehensive planning in advance of regional development.

Regional planning involves (1) an intimate knowledge of physical conditions; (2) plans for physical improvement; (3) certain regulations to insure a sound social structure; and (4) the necessary legal machinery to enforce and

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protect the plans. First and foremost, however, is the knowledge of physical conditions. The other phases of planning must accompany or follow a study of the physical characteristics of the region; hence adequate base maps for scientific regional planning are obviously fundamental.

While every region presents a totally different set of conditions, the requirements in the form of base maps will be much the same. Except in unusual instances the maximum dimension of a region will be approximately 100 miles, since modern forms of rapid transit rarely distribute people in appreciable numbers beyond a 50-mile radius. While some further improvement in air transport may quickly expand this radius, the requirements for base maps for regional planning should not be materially changed. In cities and regions without rapid transit, 20 miles appears to be the maximum radius of distribution using the automobile and ordinary surface transit. This means a maximum dimension of approximately 40 miles to be considered in the adoption of a base map. A 50-mile radius, or even a 20-mile radius, covers a large area. In most regions the area will not be circular, because of the topography—such as the ocean at San Francisco, Calif., Lake Michigan at Chicago, Ill., or the mountainous areas about Los Angeles, Calif. These physical features greatly reduce in size the actual areas to be dealt with in any detail. The extent of the areas to be covered, however, produces problems in mapping from the standpoint of cost and of securing accurate information and charts of usable sizes.

The purposes for which base maps are needed in regional planning may be broadly defined as follows:

1.—For general portrayal of existing physical characteristics.
2.—For basic information, field and office study.
3.—For accurate designing and cost estimating.

It is suggested that the scales of these maps should be, respectively, 2 miles to the inch, 2 000 ft. to the inch, and 200 ft. to the inch.

Regional planning that is scientifically undertaken, as distinguished from that which is piecemeal or opportunist, depends on a broad knowledge and understanding of the conditions. A map at a scale of 2 miles to the inch should show water areas, rivers, and streams, low-lying or swamp lands, forests and wooded tracts; also elevations by 25 or 50-ft. contours, political boundaries, existing facilities for communication (roads, highways, streets, railroads, and transit lines) and broad uses of land (agriculture, parks, reservations, oil fields, water-sheds for domestic water supply, etc.). At this scale a map of maximum dimension of slightly more than 4 ft. would be required for a region of a 50-mile radius; and one of less than 2 ft., for a region of 20-mile radius. Such maps are usable. They would be of service primarily as a basis for illustrative drawings and diagrams, such as a relief map showing physical conditions and topography; a map showing areas dominately suited for urban occupancy, agriculture, or open space reservations; a map showing population distribution, a broad scheme of transportation, including rapid transit facilities, streets, and highways; and similar studies. Regional planning, like all other great public movements, requires public understanding and support.

This small map would be highly useful for the reproduction of studies in thoroughly legible manner. There need be no particular sacrifice of accuracy or attempt at precise measurements in such drawings, although on the other hand there need be no deliberate distortions of scale in fact presentation.

The map to a 2 000 ft. scale should contain all the information previously suggested for the smaller size. Increased accuracy of detail and of delineation of specific areas, such as swamps and forests, of stream alignment, and of closer contour elevation would be possible. Except where grades are unusually steep the contour interval should be approximately 20 ft.

This is the principal base map to be used in regional planning. It would have a maximum dimension of approximately 22 ft., for a region of 50-mile radius, and a maximum dimension of 8 ft. for a region of 20-mile radius. Because of the large size, it would have to be made in sections as is done by the U. S. Geological Survey. For field work and for various office studies as many sections as might be needed could be put together for whatever use desired or for whatever section of the region might be under consideration. Full maps of the entire region at this scale would be used chiefly as wall maps.

Practically all studies would be made on this base map, such as existing uses of land, industrial, commercial, residential, agricultural and recreational; population distribution; population density; existing railroads and terminal facilities, interchange points and classification yards; existing main highways; traffic flow; freight traffic handled by railroads; passenger traffic handled by railroads; drainage districts; areas served by sewers and water; and all such factual studies showing information of value to be used in the actual preparation of plans.

Likewise, on this map will be shown preliminary plans for streets and highways, public recreation areas, transit and transportation plans, and proposed zoning. Except where very irregular topography exists, this map would serve most satisfactorily for the preliminary location of individual highways, bridges, parks, and general delineation of districts for zoning purposes.

It is surprising how few adequate maps are found in cities, and, to a much less extent, in regions where comprehensive planning has been undertaken. Regional planning would be well-nigh impossible but for the splendid maps published by the U. S. Geological Survey. In many districts these maps are quite old; or they may be of varying scale; or they may not even yet have been made. The 4 000-ft. scale, the 1 mile to the inch, are most commonly found, however. If the 2 000-ft. scale could become standard, it would be highly desirable. It has been the basis for studies undertaken by the New York, N. Y., Washington, D. C., and Los Angeles regional planning organizations. In the Los Angeles area a revision of the old maps was recently completed by the U. S. Geological Survey through the use of airplane photography with co-ordinated ground control. The result is a most useful and up-to-date map with a sufficient degree of accuracy to be entirely satisfactory for regional planning purposes. Students of regional planning should be among the foremost advocates for early completion of the entire mapping work yet to be done by the U. S. Coast and Geodetic Survey and the U. S. Geological Survey.

Regional planning authorities are not charged with the execution of planning projects. For many years at least the actual execution of work will rest with local governmental agencies, such as municipalities, counties, and special commissions, such as park boards. It is to be assumed, therefore, that a small scale map of a high degree of accuracy for the entire region will neither be available nor possible of completion for many years. The earlier such a map could be completed, however, the better it would be for purposes of scientific planning; but until regional authorities are actually charged with the execution of public work, it is not probable that small scale maps of great accuracy will be secured, except in the areas where local governmental agencies exercise jurisdiction.

A map to a 200-ft. scale with 2, 3, or 5-ft. contours should be in the possession of every municipality of any size. The City of St. Louis, Mo., has had such a map since 1895. The topographic maps of Baltimore, Md., and of Cincinnati, Ohio, are well-known examples of good base maps. Pittsburgh, Pa., is now engaged in the preparation of such map based on a precise survey with 2½ and 5-ft. contour intervals. Portions of Allegheny County are also to be included. This will prove to be a most valuable base map for regional planning purposes.

It is interesting to note that the preparation of city plans in recent years has led to the undertaking of accurate topographic base maps in numerous smaller cities, such as Schenectady, N. Y., and Evansville, Ind. Such cities will always be able to expand the surveys out into the regional areas for planning purposes.

Little has been said here of serial photographic maps. The air map is almost indispensable in regional planning. It does not give the very accurate and necessary dimensional information except when undertaken in conjunction with ground control; but it does give information that ordinary survey maps lack. The actual character of terrain, of built-up areas, the nature of wooded tracts, or of stream beds, is much better visualized through the medium of the air map. Those engaged in regional planning will find it of increasing use. Its scale will vary according to a large variety of uses to be made of it. Possibly a 500 to 1 000-ft. scale would prove most practical in the majority of cases. The oblique photograph from the air is also exceedingly useful for some purposes.

In conclusion, it should be repeated that good base maps are a first requisite of regional planning, but that very few such maps now exist in adequate form. In view of the vast amount of regional planning which will be undertaken by American cities during the next few years, more and more attention will have to be given to the preparation of adequate base maps of several different sizes, supplemented by air maps. Of particular value to regional planning work in all cities and regions are the maps prepared by the U. S. Geological Survey. Increased appropriations for that important Government Department will greatly facilitate the cause of regional planning.

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STANDARD CITY ENGINEERING SURVEYS[*]

By Robert H. Randall,[†] M. Am. Soc. C. E.

American cities need comprehensive engineering surveys as a basis for operation and planning. Since, generally speaking, fundamental conditions in all of them are very similar, it seems reasonable to assume that the survey program which proves satisfactory for one, will serve with little modification for all. The adoption of a standard program and its continued, successful use over considerable periods by numerous, widely scattered cities, would seem to establish both its need and its universal applicability. A consensus of practice in regard to the schedule of necessary survey information does exist, and this schedule is found in operation in those cities which are most completely equipped in this respect. It is the purpose of this paper to discuss this program, or schedule, to tell how it is commonly initiated, and to describe briefly its execution and uses.

The schedule of these engineering surveys will be termed the City Survey. In the meaning herein used, this includes only the physical, or engineering, surveys of metropolitan areas. It is recognized that other basic surveys are necessary to secure data on social and economic conditions, but these are beyond the scope of this paper. To be as specific as possible, only the city or metropolitan area is considered. Surveys for a region cannot be so definitely described, since its limits are dependent on interpretation, and may embrace anything from the smallest group of related communities to large areas comprising all or parts of different States.

Although an adequate definition of the city cannot be attempted here, it may be well to consider some of the city's characteristics as they apply to engineering surveys. Foremost among these is the fact that the city is a living thing, with the properties of change and growth. Whereas in the past it has grown in casual fashion by the uncontrolled addition of new subdivisions, more and more it is now growing of itself and from within, according to well-

Note.—Discussion on this paper will be closed in **December, 1928.**

[*] Presented at the Joint Meeting of the Surveying and Mapping Division and the City Planning Division, New York, N. Y., January 19, 1928.

[†] Pres. and Chf. Engr., R. H. Randall & Co., Inc., Toledo, Ohio.

ordered plans made by its own citizens for the greatest public good. Under the old process of growth by haphazard accretion, the need of plans, and especially the information on which plans were based, was not realized. The modern method of considered betterment and expansion is predicated on planning of the sort which presupposes facts. The City Survey secures and presents these facts.

The business of the modern city may be considered under two heads, namely, operation and planning. City operation, or management, is steadily becoming more business-like and scientific. It consists in the successful operation of all the community's existing facilities. Concurrently, adequate plans must always be available for the improvement and extension of these facilities as the needs of change and growth require. Both these activities—operation and planning—demand for their successful performance a full knowledge of all physical facts and conditions. This may be said to correspond to the inventory of private business, and is supplied by the City Survey.

In contrast to city planning, the City Survey, in its engineering sense, has little appeal to laymen, and is of no popular interest in its initial stages. It is of so technical a nature that it may be generally appreciated only through its results. So great, however, are its economies and advantages, that, once begun, even in a small way, it almost invariably goes on to logical completion. A small amount of tangible accomplishment serves to acquaint even lay disbursing authorities with its value, and appropriations for its continuation are relatively easily obtained.

Formerly, the City Survey was usually undertaken as the engineering basis of some specific improvement program, such as large additions or alterations to sewer or water systems. It is now increasingly the practice to undertake it as the foundation of a broad community planning program. Since it is of permanent value, it is a reasonable capital expenditure, and may properly be financed either from bonds for specific improvements or from appropriations out of general revenue.

In so far as any order of preference may be observed, the engineering survey should come first, since physical considerations profoundly affect the economic and social phases. It is certain that, if made first, it greatly facilitates the others. It is probable that many city planning reports and recommendations made in the past are, to a certain extent, discredited because of the lack of adequate physical information. Whether or not, in all cases, the engineering survey should precede general planning recommendations may be a matter of policy; but it is clear that specific recommendations and detailed plans for construction should not be undertaken without it.

Schedule.—A consensus of actual practice indicates that the City Survey comprises the following general divisions: (*a*) Triangulation; (*b*) traverse; (*c*) levels; (*d*) topographic map; (*e*) property map; and (*f*) wall map.

In some instances the major part of the City Survey schedule has been officially termed the Geodetic and Topographic Survey. Under this classification the first three items—triangulation, traverse, and levels—are known as the Geodetic Survey because of the use of methods characteristic of geodetic

surveys of large areas. The topographic mapping and the compilation of a wall map are included in the Topographic Survey.

In any consideration of a proper program of physical surveys, whether for a city or a region, it is assumed that the fullest possible use is made of the controlling, continental surveys of the United States Coast and Geodetic Survey, and the standard maps of the United States Geological Survey. With the development of the country the definite economic advantages of these Federal surveys are increasingly apparent. Where they are not yet available, every effort should be made to influence State authorities to co-operate with the Federal Government in financing their completion. The expense is small, the data furnished are not procurable otherwise, and they are of practicable value to all other engineering surveys.

The place of aerial mapping in the City Survey schedule has been the subject of much controversy. As a comparatively new method, it has received much publicity, and rather sweeping claims have been made for its results. Air maps have a distinct value in city planning and operation, but it is believed that this consists more in their use as exhibits or explanatory information than as substitutes for the surveys recommended in the foregoing schedule. They have the advantage of ready comprehension by laymen and of the presentation of a practically unlimited amount of detailed data.

The chief obstacle to the use of air maps in the City Survey lies in their limitations for scaling purposes. The air map is fundamentally a photograph, and, consequently, is subject to perspective errors. These increase rapidly with the map scale until a point may be reached where so many ground locations are necessary that the added expense of air views is not justified. Land values throughout the metropolitan area are invariably considerable, and the improvements laid out on this land are costly. It is, therefore, good business to measure and record the physical facts relating to land and its occupation on a scale which experience shows to be desirable for the purpose—usually 1 in. = 200 ft. for the parts of the metropolitan area in which development is under way or is looked for in the near future, and 400 ft. for the outlying areas as to which information is desired for sanitary or other district work. On neither of these scales will the air map ordinarily give results of as great accuracy for the same expenditure as is obtainable by ground methods.

Where scaling accuracy is not a requirement, or where small scales are used, air methods in many cases will give satisfactory results at less cost than ground surveys. Considerable use has also been made of air views in the construction of assessors' maps, on large scale; but it is to be pointed out that this and other similar uses may be classified as emergency uses, since, although the information may be obtained very quickly in this manner, it would have already been available, as a matter of course, if the surveys of the schedule recommended herein had been in existence. Air maps are to be considered as adjuncts to, rather than substitutes for, any of the various divisions of the standard City Survey.

Triangulation.—In the execution of any survey extending over a large area it is necessary to provide for the logical distribution of the small, unavoid-

able errors incident to any system of measurements. If it is attempted to join together small, individual surveys, each satisfactorily accurate in itself, it will be found that the small discrepancies between them will accumulate and give trouble; and the greater the number of surveys and the farther the distance from a central point of origin, the greater the difficulty. It is essential to plan from the whole to the part, rather than from the part to the whole. For this purpose a system of controlling surveys of an accuracy superior to the smaller detailed surveys is first established. After its discrepancies have been determined and adjusted, it constitutes a basic framework to which all other surveys may be referred. For the City Survey two controlling systems are necessary, the horizontal and the vertical.

Triangulation constitutes the chief horizontal control. Basically, it is a series of overlapping triangles. After a certain number of triangle side lengths, or base lines, have been measured on the ground, the others become known by measurement of angles. Because of the rigid geometrical conditions governing triangles, this form of survey is capable of very precise adjustment and computation of resultant positions. For the control of metropolitan areas, distances between triangulation stations are determined with a precision of from 1 part in 50 000 to 1 part in 100 000.

The arrangement and distribution of triangulation stations necessarily vary with the conditions encountered. Ordinarily, an average of one station for every 1 to 3 sq. miles is satisfactory. All stations are monumented and referenced in as permanent a manner as possible.

Traverse.—Traverse lines, executed with transit and tape, and with almost the same procedure as the measurements of triangulation base lines, are used to extend and to make more convenient the results of the triangulation. Traverses originate and close upon triangulation or upon previously adjusted traverses. They are run over streets or highways, and the stations thus established, are carefully monumented and referenced.

Under some conditions the traverse itself constitutes the chief horizontal survey, without triangulation. Results of an accuracy averaging 1 part in 30 000, or better, may be expected from it.

Levels.—Levels are classified as first and second order in respect to accuracy, and constitute the vertical control system. Levels of the first order are run by the most precise methods; are then double-run, and the backward and forward runnings of each section must not have an error (in feet) greater than $\pm\ 0.017\ \sqrt{M}$, in which, M is the length of the section, in miles. Permanent monuments, or bench-marks, are established in pairs at proper intervals throughout the survey area. Levels are run in loops or circuits, and the closing errors of these are distributed by an adjustment according to the method of least squares. Leveling of the second order of accuracy originates with and closes upon the positions established by the first-order system. Its monuments may be less permanent in character, it is single-run, and is required to close within $\pm\ 0.05\ \sqrt{M}$.

Topographic Survey.—The topographic survey comprises the making and publication of topographic maps of the metropolitan area. The purpose of

such mapping is too well understood to require emphasis. Briefly, it records and presents in the most business-like fashion all the essential information pertaining to the shape, elevation, and other physical characteristics of the land surface, and the properties, structures, and other improvements laid out on it. Such maps bring, for all practical purposes, the terrain itself to the engineer's or planner's desk. Their use eliminates all ordinary preliminary surveys and supplies most of the needed data for construction plans. In addition to their value as the basis for designing improvement projects, the maps constitute an authoritative reference for all questions regarding land conditions that may arise in the conduct of the community's business. Properly executed, they form a permanent inventory of the two great physical considerations, land and its occupation, and constitute what is perhaps the best known division of the total City Survey.

Actual mapping is best done by the plane-table method, controlled, of course, by the co-ordinating positions of the preceding precise horizontal and vertical surveys. The maps of the important parts of the metropolitan area are on a scale of 1 in. = 200 ft. The individual sheet size is commonly about 20 by 25 in., the longer dimension being east and west. For large areas the exact sheet limits are based on even multiples of seconds of latitude and longitude, usually 35″ or 40″ of latitude and 60″ of longitude. This is desirable because it permits a combination of plane and spherical projection, which tends to reduce the amount of variation from true azimuth incident to any considerable extension of a plane co-ordinate system.

The field sheets for plane-table mapping are mounted on metal, to insure retension of scaling accuracy despite changes in humidity. Each has plotted upon it, before it is taken into the field, all positions of triangulation and traverse, and all level elevations within its area. Using these data as a basis, the field party records, by plane-table and stadia methods, all details of topography and drainage; streets, alleys, and public property lines; railroads, bridges, tunnels, and retaining walls; public and industrially important buildings; dwelling-houses in unsubdivided territories; property and survey monuments; wooded areas; etc. The exact amount and detail of information desired necessarily vary somewhat in different instances.

After the map is completed in the field, all the street and public property lines are carefully compared with all existing records, thus providing an additional check on the accuracy of the map. Formerly, after this checking was completed it was the practice to trace the penciled lines of the field sheet in inks of various colors before proceeding with publication. This step is now usually eliminated, and the reproduction of any number of copies is accomplished without changing the condition of the field sheet, thus leaving it in shape to be taken back to the field at any time for revisions or additions.

In order both to make a large number of copies available for use and to preserve the original field sheet from deterioration through use, it is necessary to reproduce or publish copies of the topographic map sheets. This is now done chiefly by combined photo-lithographic and engraving processes. The final copies show topography, by means of contours, in brown; drainage, in blue; property lines, structures, and lettering, in black; and wooded areas

and in some cases, public property, in green. It is essential that all the accuracy of the metal-mounted sheets be preserved in the press-plates from which final copies are printed. For this purpose large and precise camera equipment is necessary, and it is advisable that the boundaries controlling the plane-projection plotting of the field sheet be drawn on each sheet by a standard metal template by which the focusing marks on the ground glass are also established.

Property Survey.—This survey includes the preparation of all those maps which require the showing of figures and dimensions. For this reason the scale is usually about 50 ft. to the inch. The map sheets are usually laid out to conform to the topographic survey sheet system. Controlling and coördinating positions are furnished by the triangulation and traverse, and all other field work is accomplished by careful transit and tape traverses. The base maps thus constructed, serve not only for property records, but for all other maps which require the showing of numerous details, such as those used for recording the positions of underground structures, construction plans for sewer and water improvements, etc.

Accurate and reliable property information is the basis of this survey, and the work of securing it may be considered as two separate steps. The first of these is the determination of the actual location, on the ground, of all street corners, angles, and curve points. The second step is the precise measurement and co-ordination of these points. It is sometimes the practice to employ an outside organization to execute the second, or precise survey step. The first, however, should be performed by the best informed local surveyors, who should possess more intimate and reliable information regarding the actual location of property lines than is possible to outsiders.

It is becoming increasingly the practice to begin the complete schedule of the City Survey by establishing controlling monuments in the outlying, or development, sections, for the purpose of tying-in new subdivisions. Under existing platting laws it is frequently possible to require new subdivisions not only to submit surveys of their own boundaries with prescribed accuracy and to monument street corners properly, but also to tie in to the controlling points established by the city. If this procedure is carried out, a complete and coördinated property map of the newer parts of the city will be built up without cost to the public beyond the establishment of the few controlling points and the necessary supervision required in co-ordinating the allotments with them.

Wall Map.—The purpose of the wall map is to present a general view of the entire area. Its scale, therefore, will necessarily vary with the extent of the territory to be covered, since the map itself should be of a size suitable for desk or wall use. The wall map is compiled chiefly from the maps of the topographic survey, usually to a large extent by photographic reduction. Some copies should be made available showing all the colors of the standard topographic survey reproductions, and some should be prepared without the topography, showing the black, or the black and blue, only.

Summary.—The City Survey, as outlined, is the common-place and common-sense physical basis for the conduct of the city's business. Without

publicity value or popular appeal, its need is yet so fundamentally obvious that all American cities will eventually possess it. With the increase in the fact-finding, scientific methods of engineering design, and the growth of the city and regional planning movement, cities are increasingly preparing themselves with such surveys. Land being the only thing on which to build, and building being more and more complex and costly, it is obvious and inescapable that some day every community will possess this fundamental knowledge of the land and its occupation.

The uses of the City Survey, as they relate to city operation, are practically as many and varied as the list of the city's activities. The application to planning is no less complete. Practical planning may be divided into two classifications: (1) That which occurs within the built-up area and may be called corrective, remedial, or re-planning; and (2) that which is undertaken in the development areas, in advance of actual occupation and construction. Here, in the city of to-morrow, is perhaps the greatest opportunity for constructive planning. Upon the topographic maps of the City Survey a street layout, or physical framework, may be determined with the greatest degree of probable future satisfaction; and when this is done, a large part of present-day city planning will have been provided for in advance. Planning in advance of development, and in accordance with the public interest rather than the narrower wishes of the individual land owner, is very much the public's business, and will undoubtedly be more and more the practice as time goes on. The National Conference on City Planning and the National Association of Real Estate Boards have gone on record as favoring the laying out of main thoroughfares in outlying districts by planning commissions, in accordance with master plans. Such plans, of course, can only be made on full information, such as is provided by the City Survey.

Some years ago the following series of questions was put to an eminent planner:

1.—Is the standard City Survey (as outlined herein) satisfactory as a basis for city planning?

2.—Should the City Survey precede the City Plan?

3.—Assuming that the Survey is necessary, and should precede the Plan, what is the best method of bringing this ideal procedure into every-day use?

The answer was: That the Survey was the basis for planning; that the Survey should precede the plan; and that this procedure should be brought into being "in the usual manner". The writer has never discovered exactly what was meant by "the usual manner"; but it seems likely that the consideration of the allied subjects of planning and of basic engineering surveys, jointly by the two Technical Divisions of the Society most directly concerned, will lead to better understanding of the problems involved and to better practice in city surveying and city planning.

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PAPERS AND DISCUSSIONS

FINAL REPORT OF THE COMMITTEE OF THE STRUCTURAL DIVISION ON FLORIDA HURRICANE*

In enumerating the effects of the Florida hurricane of September 17 and 18, 1926, it may be well to give a brief description of the storm. As in all tropical hurricanes, the wind blew in a circle with an area of calm in the center. F. L. Stearns, Assoc. M. Am. Soc. C. E., who visited Miami on behalf of the Committee soon after the storm, stated that the first half came from the northeast and the second half from a little east of south instead of from an exactly opposite direction. He felt that the explanation for this was that the storm changed form somewhat when it struck the land. He assumed that the center of the storm passed directly over Miami. It can, however, be readily explained by assuming that the center of the storm passed somewhat to the south of Miami. In Fig. 1 is shown a diagrammatic sketch of the storm. Point *A* represents the location of Miami during the first half and Point *B*, the location for the second half. The arrows indicate the direction of the wind. The time between the first half and the second half was about an hour, which indicates that Miami was not far removed from the path of the storm center.

In his report Mr. Stearns gave the storm effects on all the various forms of structures, but as a large part of these were not designed by engineers and are of such a character that they are not susceptible of close mathematical analysis, the Committee can add very little to present knowledge by a study of them. As the damage done to these structures was so many times greater than that done to structures designed by engineers, the necessity of technical design or more careful construction for the smaller structures as well as the larger ones is clearly shown.

It would be of considerable value to the Engineering Profession if an accurate determination of the wind pressure could be made, but the Committee does not deem it possible. On the top of the Meyer-Kiser Building was a small observation room, about 18 by 21 ft., with four small columns in the corners supporting the roof. The walls were blown out and the four columns

* Presented at the meeting of the Structural Division, New York, N. Y., January 19, 1928.

bent. It would require a pressure of about 65 lb. per sq. ft. to produce the distortion which occurred and this figure is probably as good as can be determined; but it cannot be taken too seriously because the area of exposure at the time of failure is uncertain.

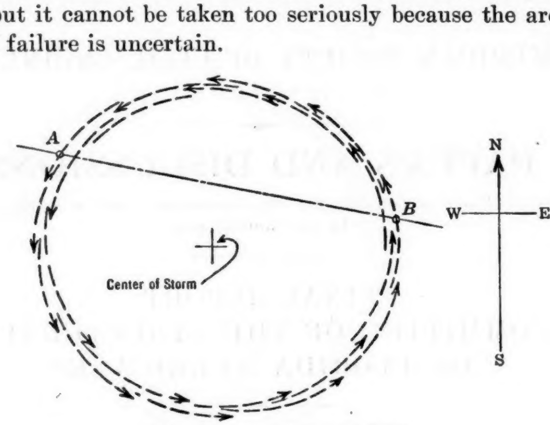


FIG. 1.—DIAGRAM OF STORM.

Professor Morris, of the Committee, has computed that it would have required about 60 lb. per sq. ft. of wind pressure to bend the columns in the front of the Meyer-Kiser Building. It is possible that the failure of the wall allowed the building to sag quickly to leeward and applied a blow of unknown force to the columns, producing a serious effect, although the wind, striking the building at an acute angle, may not have applied a very severe normal pressure. Hence, the static stresses computed may have little relation to the applied pressure. The evidence, however, indicates that for areas of moderate size, the storm probably exerted a pressure of more than 50 lb. per sq. ft., but there is no evidence that this was the case over the entire area of a building.

The two Miami buildings which suffered most from the storm were the Meyer-Kiser Building and the Realty Board Building. A study of the design of these two buildings teaches some interesting lessons. The plan of the Meyer-Kiser Building is shown in Fig. 2 (*a*), from which it can be seen that the south end is about 45 ft. wide and the north end about 39 ft. wide. The length is about 140 ft. The height was about 176 ft., not including pent-houses. The dashed line shows the distortion from the storm. On Fig. 3 is shown a typical floor plan. The type of wind braces used is shown on Figs. 2 (*b*) and 4. The connection shown is used at points marked *B* in Fig. 3. Based on the number and size of the rivets in the wind braces and assuming the elastic limit of the steel at 36 000 lb. per sq. in., the steel work alone, without any help from the walls, should have resisted a wind pressure of about 15 lb. per sq. ft. at the fifth floor. However, the clip angles were so thin that in bending they did not develop more than 25% of the value of the rivets. These clip angles in many cases bent or broke, thus destroying all the necessary strength; and even if they had not broken they would have been so limber as to have given the building little stiffness. The columns too were not suited to develop the strength for the type of connections used.

The columns are made up of plate and angle sections and in the area that failed they had few cover-plates. The thickness of the angles in the columns was such that the outer rivets connecting the clips to the columns could not develop their full strength before bending the flange angles of the columns.

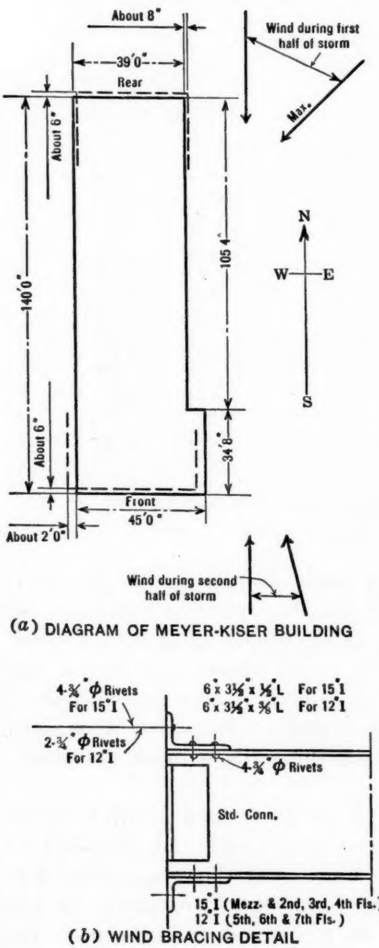


Fig. 2.—Plan and Details, Meyer-Kiser Building.

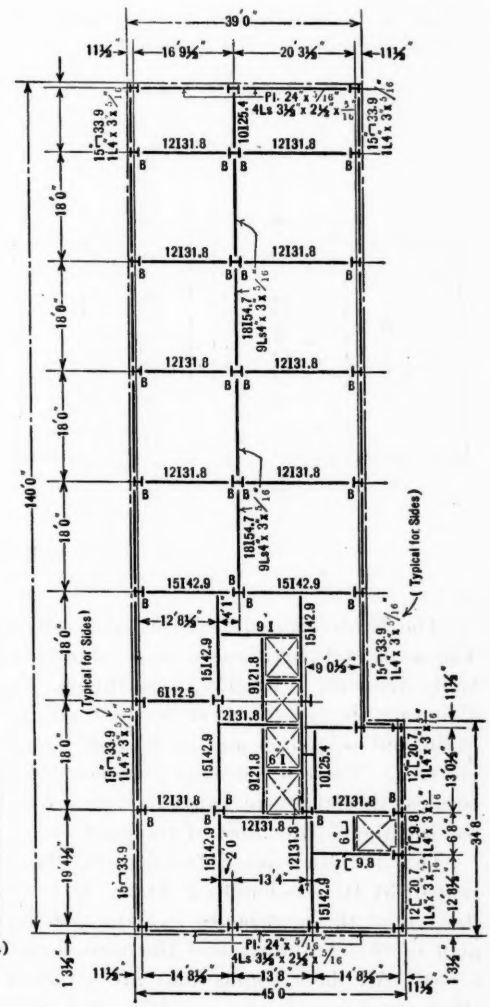
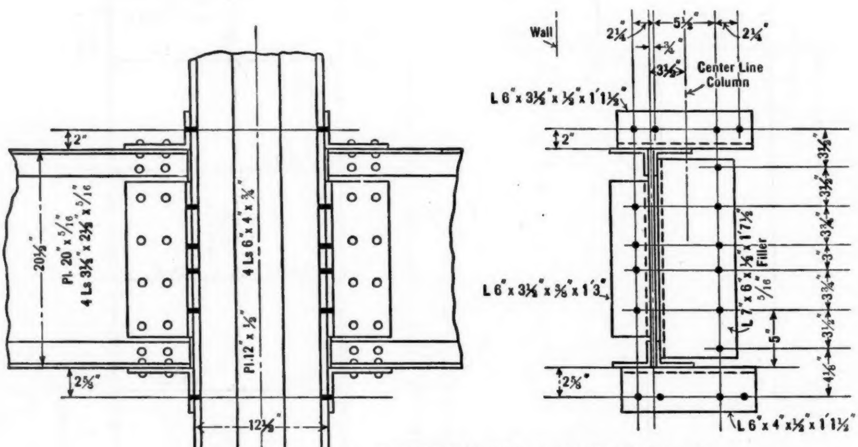


Fig. 3.—Typical Floor Framing Plan, Meyer-Kiser Building.

In addition, in the south end of the building, the floor system was broken up with elevator shafts and there was very little bracing. What bracing there was, was on beams shallower than the 24-in. spandrel girders in the front or south wall. The result was that this wall took nearly all the wind load for this end of the building until it failed. At the north end there was bracing

of a similar character, but more bracing in the floor system, which evidently is the reason for less distortion of this end of the building. In conclusion, it is perfectly evident that there was no adequate wind bracing in this building, and the bracing which was used, was made valueless by the details. It is difficult to understand how any loans or hurricane insurance could have been obtained on this building. The results bring out forcibly the desirability of engineering advice for financial organizations as well as for the owners.



COLUMNS 10 AND 11 AT THE 7th FLOOR

FIG. 4.—WIND-BRACING DETAIL FOR MEYER-KISER BUILDING.

The Realty Board Building also suffered quite severely during the storm. The walls and partitions were much cracked, but the steel framework was not badly distorted. Mr. E. A. Sturhman who was engineer in charge of reconditioning this building, states that a careful survey shows that it was only 3⅞ in. out of plumb and part of this may have been due to inaccuracies of erection. There was greater evidence of lack of stiffness than lack of strength, because the structure swayed enough to crack the walls without, seemingly, passing the elastic limit of the steel.

This building has a frontage on the west of 45 ft. and a width in the rear of 34 ft. Its length is 94 ft. It is fifteen stories high and the height of the top of the pent-house is about 166 ft. Fig. 5 (*a*) shows a typical floor plan of this building, and the main bracing connections are shown in Fig. 5 (*b*). All these connections are attached to 15-in. and 18-in. beams. The other transverse beams are 6 in. and 10 in. deep in the typical floors, with very much less rigid connections. It is probable that, on account of their greater stiffness, the beams and connections shown in Fig. 5 (*b*) took practically all the wind stresses. The connection of the wind brackets to the columns is an important feature of the construction. The second line of rivets connecting the brackets to the columns is practically valueless because the connection angles of the thickness used would be weak in bending at the root if the full strength of these rivets was developed. Mr. Stuhrman stated that

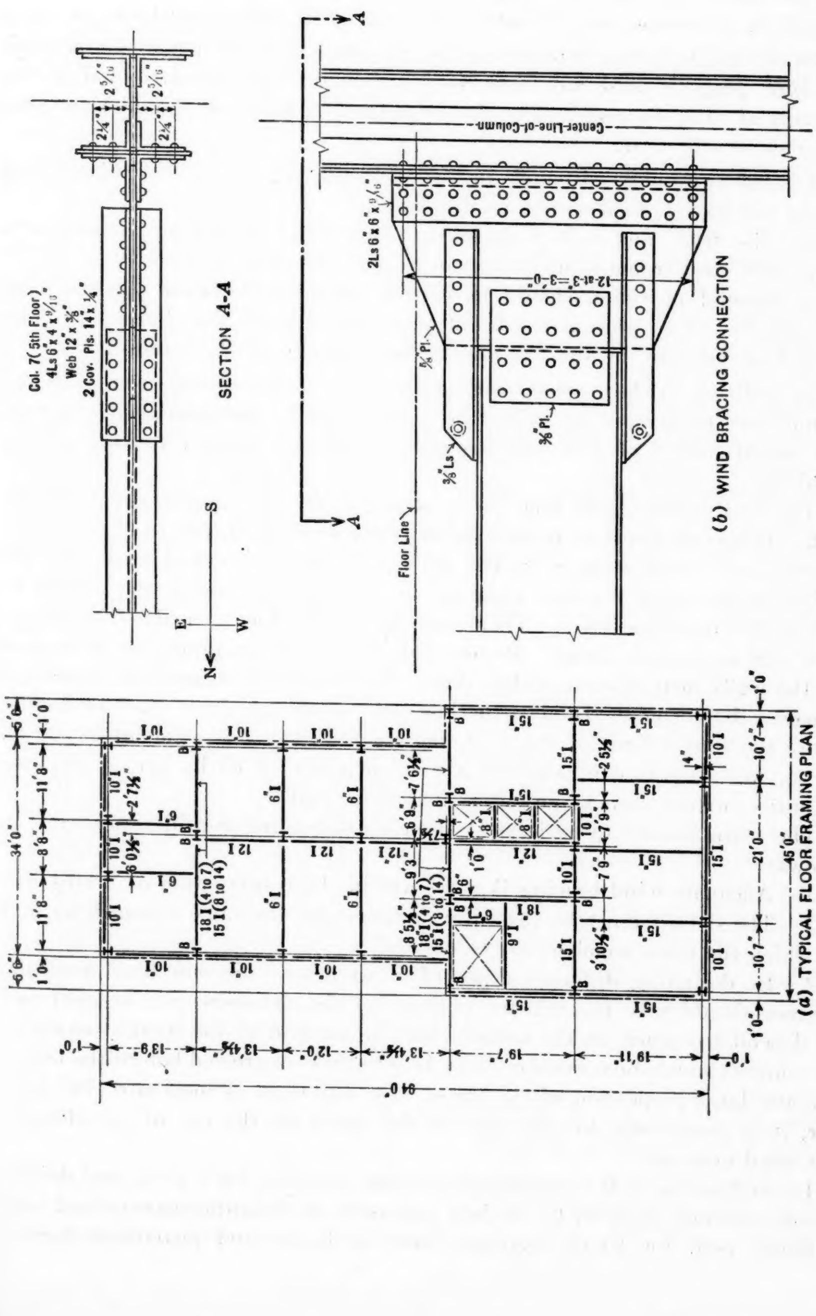


FIG. 5.—REALTY BOARD BUILDING DETAILS.

the steel work was checked and found to comply with the Building Code which calls for a 20-lb. wind load. Probably the combined stresses used were 24 000 lb. per sq. in. Without entering into any controversy on this point, the Committee doubts whether the available strength (before a 24 000-lb. unit stress was reached in some parts) would withstand a full 20-lb. wind load. A study of the design, however, indicates that a genuine effort was made to brace this building properly and any shortcomings should not be attributed to an attempt at false economy. The following points, however, should have been given more attention:

1.—The two unbraced panels in the rear were an element of weakness and should not have been designed in this manner.

2.—The front and rear walls should have had stiffer bracing because a fairly stiff steel frame is necessary to prevent cracking of walls.

3.—Special provision should have been made at the stair opening near Column No. 14 to prevent the beam between Columns Nos. 14 and 15 from being bent laterally by the twisting of the rear end of the building.

In addition to these considerations, in both these buildings some attempt should have been made to stiffen the building in a longitudinal direction as this would have had a strong tendency to resist torsional racking of the building.

The Daily News Building has a tower 40 ft. square and about 255 ft. high. It was designed to resist a wind pressure of 20 ft. per sq. ft. in either direction at a unit stress of 24 000 lb. per sq. in. in the steel work. In the design of the wind bracing, care was used in proportioning the details as well as the main members. There was a special effort to make the connections stiff as well as strong. Besides this, heavy interlocking tiles were used for the walls instead of partition tiles. This building suffered no structural damage. Based on these facts, the building should not show signs of failure before the elastic limit of the steel was reached, which is at about 36 000 lb. per sq. in. This is equivalent to a wind pressure of 30 lb. per sq. ft. over the entire surface without any assistance from walls.

The main lessons to be learned from the foregoing considerations are as follows:

1.—Adequate wind bracing is necessary in the construction of a building.

2.—The details must be carefully designed for the same strength as that for which the main members are calculated.

3.—In designing different types of wind braces for one floor level the designer should take the relative stiffness of the members into account and not depend too much on the arithmetical summation of the total strength of the connections as some members may be greatly overstressed before the others take any large proportion of the stress. On buildings of moderate size, however, it is permissible to take care of this point by the use of an adequate unit wind pressure.

4.—Stiffness as well as strength is a prime requisite for a good steel design. This is especially realized by the best engineers in designing narrow and high buildings used for living quarters, such as hotels and apartment houses,

where the confidence of tenants in the stability of the structure is of the utmost importance.

5.—The effects of the Florida hurricane indicated that the floor system acted as a stiff plate or horizontal girder and that all columns were subject to the same horizontal deflection, except that the entire structure twisted, where one end of the building was stiffer than the other. In designing wind bracing this must be given full weight and the bracing arranged, if possible, so that no twisting action from the wind can come into play. This means that the wind pressure at one end of a building and the bracing designed to resist it, must balance the relation between the corresponding pressure and bracing at the other end, so that the structure will deflect about the same amount at both ends. This principle applies especially where one end is higher or narrower than the other.

The following is quoted from a preliminary report made to the Structural Division on January 20, 1927, by the Chairman of this Committee:

"Summarizing the fields that this Committee might well cover are the following:

"1.—A determination of the wind pressure exerted by the storm.

"2.—A standard method for figuring wind stresses in tier building construction.

"3.—Standard details best able to resist wind stresses both from the standpoint of stiffness as well as that of strength.

"4.—A study to determine whether the same requirements for wind should be used for buildings having a low ratio of height to width as for buildings having a high ratio.

"5.—The strength of masonry walls and the bearing they have on wind bracing.

"I feel that there is a good chance of our accomplishing something valuable in connection with a part of these at least though it may entail more work than the Committee as now constituted can find time to perform. I am sure, however, were we able to cover any one item thoroughly, our work will have been well worth while."

Taking up the items mentioned in the order given:

1.—A Determination of the Wind Pressure Exerted by the Storm.—The Committee feels that, unfortunately, a strict determination of the wind pressure cannot be made. This has been covered earlier in the report. The results showed, however, that all the tall buildings consistently designed for a wind load of 20 lb. per sq. ft. survived without serious damage.

2.—A Standard Method of Computing Wind Stresses in Tier Building Construction.—The results of the storm showed that for buildings of the size and shape found in Miami, the common theory, as given in Fleming's "Manual on Wind Bracing", will produce structures capable of standing up under the most severe conditions. The failures were caused by neglecting to apply the theory consistently. On the other hand, buildings so designed will not necessarily be sufficiently stiff to give the tenants confidence in their safety. Moreover, in the case of very high or narrow structures, the obvious errors involved render the results less secure than is desirable. From a standpoint of strict theoretical determination of wind stresses, an adaptation of the slope deflection

method will probably give the best results.* Some of the members of the Committee, however, are not entirely convinced that this method can come into universal use on account of the work that it will entail.

A consulting engineer generally has a limited time in which to design a structure, and if a method could be developed which will approach in results the slope deflection method and still entail very much less work, it would be desirable. One of the members of the Committee has used a method apportioning the amount of wind shear taken by the various columns on the basis of the relative moments of inertia of the columns. This is based on the assumption that the length of the column is the same. If the lengths between wind brackets vary considerably, as in the case of deep girders, these shears will have to be corrected to take care of the difference in column lengths. A closer approximation would take into account the effect of variations in the stiffness of attached floor-beams, which would involve practically the method developed by Professor Morris. The Committee feels that some study should be given to this before suggesting a definite method for computing wind bracing.

3.—Standard Details Best Able to Resist Wind Stresses Both from a Standpoint of Stiffness as Well as That of Strength.—Such details have been pretty well standardized by a few of the leading consulting engineers in New York, N. Y., and these could be released at any time. It would seem advisable, however, to release such information at the same time that a method for computing wind bracing is developed and, therefore, no action has been taken in regard to this subject.

4.—A Study to Determine Whether the Same Requirements for Wind Should Be Used for Buildings Having a Low Ratio of Height to Width as for Buildings Having a High Ratio.—This item is also related to the method of computing wind bracing and can well be taken up at such time as Item 2 is considered.

5.—The Strength of Masonry Walls and the Bearing They Have on Wind Bracing.—To secure any really valuable information on this subject, it would probably be necessary to have numerous tests made. This would entail considerable expense and it is doubtful whether it will be possible for the Committee to finance this without some assistance from the Society.

In the preliminary report before quoted, it was stated that to make a complete report might entail more work than the Committee, as now constituted, can find time to perform. To make an investigation along this line requires a very large amount of work, and few men are able to devote sufficient time to get results which entirely satisfy them. One of the members of the Committee, Mr. Miller, feels that a great deal of information as to wind pressure could be secured by means of self-registering barometers. He feels sure that he can secure the use of these barometers gratis, but, of course, some one would be required to keep the results and interpret them.

When the American Insurance Union Building was built in Columbus, Ohio, due to the initiative of Professor Morris, numerous gauge points were

* This is covered by the paper entitled "The Design of Tall Building Frames to Resist Wind", by Albert W. Ross, Jr., Jun. Am. Soc. C. E., and Clyde T. Morris, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1395.

established on the columns and targets were located in the elevator shafts by which it is possible to measure the lateral displacement of the building during a storm for a height of 34 stories. No doubt a great deal of valuable information could be obtained by careful readings made on this building. Generally, owners are afraid to have any such tests made as they think it will cause the public to doubt the safety of the buildings; but in this case as the matter was all arranged when the building was constructed and the public now understands that work such as this is for educational purposes, the ordinary objections do not prevail.

The American Institute of Steel Construction, for several years has endeavored to advance knowledge along engineering lines in the use of steel. This question has been discussed with some of the members of the Institute, and it is possible that if its finances permit, it could be persuaded to establish a fellowship in Ohio State University to study the entire problem of wind bracing. In this case the Institute would probably be glad to have the work done under the general supervision of a committee of the Structural Division of the Society, provided that this would be agreeable to the authorities of the Ohio State University. Such supervision, of course, would involve some expense to the Society. In all probability, the student appointed to this fellowship would work under the direct supervision of Professor Morris, of this Committee. If this should be done, many new facts might be discovered in regard to wind bracing which are now unknown, and a good student devoting his entire time to the subject under the proper supervision, could carry out many investigations which would entail far more work than any individual member of the Structural Division could devote to the subject.

In conclusion, the Committee hopes that this incomplete report may be of some value to the Engineering Profession in the design of wind bracing. It wishes, of course, to stress the point that no system of rules, however complete, can take the place of thorough technical knowledge and long experience and judgment in work of this character; and that the work of designing wind bracing for tall narrow buildings should never be entrusted to novices.

H. G. Balcom, *Chairman*,
D. C. Coyle,
F. R. McMillan,
Lee H. Miller,
Clyde T. Morris,

January 19, 1928. *Committee.*

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

PRECISE WEIR MEASUREMENTS

Discussion*

By Messrs. Th. Rehbock and Erik G. W. Lindquist.

Dr. Ing. Th. Rehbock† (by letter).‡—Accurate and reliable measurements are among the most important requirements in the field of experimental hydraulics. In the field of practical hydraulic engineering, discharge measurements which are correct within 1 to 2% are entirely satisfactory. In field measurements in time of floods, errors of from 2 to 5% and even more are unavoidable and inconsequential. Stream flow measurements are generally made by means of current meter gaugings, although weirs are sometimes used to gauge the flow of the smaller streams.

Hydraulic Gaugings by Means of Sharp-Crested Weirs.—In experimental work the engineer is confronted not only with the problem of measuring accurately the quantity of water, but, in general, he also faces the problems of regulating the flow in such a manner that a given quantity of water is discharged in a unit of time. Weirs are particularly useful gauging devices in case a predetermined rate of discharge is to be maintained, for it is necessary only to fix the water level above the weir crest at an elevation which can be calculated in advance.

Weir gaugings, because of their simplicity and their accuracy over a wide range of discharge, are better adapted for use in the laboratory than any other method that has been proposed to the present time.

In the writer's opinion, the best weirs for use in field measurements are nearly triangular in cross-section with a sloping down-stream face and with rounded (preferably cylindrical) crest. This type of weir is most satisfactory because it is stable, it is not easily damaged, and its accuracy is not affected by floating material in the water. In the laboratory, however, where the factors just mentioned are of no consequence, the sharp-crested weir is to be preferred because of the greater precision of discharge measurements.

* Discussion of the paper by Ernest W. Schoder, M. Am. Soc. C. E., and the late Kenneth B. Turner, Esq., continued from May, 1928, *Proceedings*.

† Prof., Technische Hochschule, Karlsruhe, Germany.

‡ Received by the Secretary, February 8, 1928.

Of the sharp-crested weirs, the rectangular suppressed weir with a horizontal crest, built between vertical side walls, is particularly suitable, if the jet is fully aerated and not affected by back-water. In a few cases the triangular weir (Thomson weir) may be more satisfactory, if very small rates of discharge are to be measured. It has the disadvantage, however, that the head-discharge relation is affected to a certain extent by the shape of the crest and by the size of the approach channel in a manner that cannot be clearly determined.

The discharge over a vertical suppressed weir with a horizontal crest, as shown by experiments at Karlsruhe, Germany, is exactly proportional to the length of the crest. The rate of discharge, q, per unit length of crest is, therefore, merely a function of the two variables, h_0, the head of the weir, and p, the height of the weir crest above the bottom of the approach channel, if the following conditions are satisfied:

(*a*) The velocity in the approach channel should be as nearly the same at all points in a cross-section as practicable.
(*b*) The head should be large enough so that the jet is entirely free and does not cling to the weir plate.
(*c*) The jet should be fully ventilated so that the pressure in the space between the lower nappe and the weir plate is practically the same as that in the open air.
(*d*) The water level below the weir should be so low that the heads are not affected by back-water.

Observations made in the Karlsruhe Hydraulic Laboratory, extending over a period of more than twenty-five years, have shown no measurable fluctuations in the head on a given weir for a given constant rate of discharge, if these conditions were satisfied.

Although widely different coefficients of discharge, μ_0, have been reported by various observers for presumably similar conditions, it is the writer's opinion that discrepancies of this kind are traceable either to inaccuracies in measurements or to unsatisfactory arrangement of the testing equipment. If, for a given constant rate of discharge, fluctuating heads are observed which are not due to errors in measurements, it must be assumed either that eddies that affect the flow are formed in the approach channel, or that the jet is not properly aerated and the nappe is depressed. The most common cause of disturbances in the flow over a sharp-crested weir, particularly under high heads, is inadequate aeration of the jet. These disturbances occur if the pipes which supply the air below the jet are too small, or if they are partly closed by water. In this case the μ_0-values are too large. It is the writer's experience that a sharp-crested, vertical, suppressed weir with adequate provisions for fully aerating the jet at all times, built between parallel side walls, is the best measuring device for hydraulic laboratories in spite of the fact that the necessity of adequate aeration adds difficulties to the construction of the weir.

The experimental layout should meet the following requirements if accurate measurements are to be assured:

1.—The approach channel should be of sufficient length and of the same width throughout.

2.—The water should enter the approach channel axially.

3.—The approach channel should be elevated high enough above the floor that the necessary measurements can be made easily and accurately, and it should be accessible on all sides so that any leaks in the channel can readily be detected and remedied.

4.—The approach channel should have a horizontal bottom and vertical, plane walls, the latter preferably made of plate glass.

5.—The up-stream face of the weir plate should be a vertical plane. It should have a horizontal crest 0.25 mm. to 1 mm. in thickness.

6.—The ventilation of the space under the jet must be adequate so that the water level above the weir is not affected by partial vacua under the jet.

7.—An auxiliary hook-gauge (with vernier scale) should be provided near the up-stream face of the weir plate to be used in transferring the height of the crest to the master gauge, that is, for determining the "zero reading" of the master gauge precisely.

8.—The master gauge with vernier scale which is used to determine the heads on the weir, should be above the draw-down curve of the water surface, or at a distance above the up-stream face of the weir at least four times the maximum head to be measured. The master gauge may be either a needle-gauge in the approach channel, or a hook-gauge in a special chamber connected with the approach channel.

9.—A sensitive float-gauge should be provided in a chamber built at one side of the approach channel to permit the reading of very small fluctuations of the water surface to fractions of 0.1 mm.

A careful and skilled observer is able to make discharge measurements with a high degree of precision if he has a measuring apparatus which fulfills, substantially at least, the requirements just enumerated, and if he uses a reliable weir formula.

The duties of the observer in making weir measurements are:

1.—To determine the length, b, of the weir crest. An accuracy of 1/2 000 is desirable.

2.—To determine the height of weir, p, above the horizontal bottom of the approach channel. An accuracy of 1/200 is sufficient.

3.—To determine the "zero reading" of the hook-gauge, or that reading for which the point of the hook is at the level of the weir crest. This determination involves the following operations:

(a) Bringing the point of the auxiliary hook-gauge to the elevation of the weir crest with the aid of a spirit level, and noting the gauge reading.

(b) Bringing the points of the auxiliary gauge and the master gauge simultaneously to a motionless water surface below the level of the weir crest and noting the readings on both gauges.

(c) Calculating from these measurements the "zero reading" of the master gauge, or the reading when the point of the master gauge is at the level of the weir crest.

This determination of the zero reading of the principal hook-gauge is of great importance and should be made before and after each series of tests. The precision of every measured head depends on the precision of this measurement.

4.—To determine the head on the weir by the master gauge. The point of the gauge is brought to the surface of the water in the approach channel or to the surface of the water in a chamber at

one side which is connected to the approach channel by a pipe. The head, then, is the difference between the gauge reading and the zero reading.

5.—To watch the entire gauging layout very carefully at all times in order to detect any disturbances at once and to remove the causes immediately.

After the zero reading of the principal hook-gauge has been determined, it requires only a fraction of a minute to determine the head on the weir for a given quantity of flow. A skillful observer can soon learn to make these measurements quickly and accurately. Often repeated observations have shown that the heads can readily be measured within 0.1 to 0.2 mm. An exceptionally skilled observer can even, in many cases, check his measurements by repeated observations within 0.05 mm. In the case of larger quantities of flow, however, discrepancies of a few tenths of a millimeter may occur, since the water level does not remain perfectly quiet.

Formulas for Calculating the Discharge Over Sharp-Crested Suppressed Weirs Without End Contractions.—The determination of the head of water on a weir is relatively simple if one has a well-arranged layout and accurate apparatus. On the other hand, the derivation of a reliable and accurate formula for calculating quantities of discharge from these measured heads is very difficult indeed.

Since the time of Poleni and Dubuat, who in the Eighteenth Century attempted to solve the problem of flow over weirs, many engineers have spent years of careful study and experiment to determine accurate formulas for the flow of water over sharp-crested weirs. The problem is a particularly difficult one, because the basic experimental work which includes volumetric measurements, must be quite precise, and because an empirical equation with three variables, h_0, p, and q, must be found, which is in substantial agreement with the results of the test.

All well known formulas which have been used in the past are of the type proposed by Poleni and Dubuat, namely:

$$Q = \frac{2}{3} \mu_0 \sqrt{2\,g}\; b\, h_0^{\frac{3}{2}} \qquad \text{(7)}$$

This formula is based on the fallacious assumption that water flows through the entire cross-section, $F = b\,h_0$, over the weir crest, whereas the actual cross-section of the jet, because of the draw-down which is equal to approximately $0.15\,h_0$, is only about $F' = 0.85\,b\,h_0$. In the derivation of this formula a pressure distribution within the jet itself is assumed, which is altogether different from that which exists in the actual jet. The fact that, in the case of a sharp-crested weir, the nappe rises to a point about $0.11\,h_0$ above the weir crest is neglected entirely. Despite the illogical assumptions on which Formula (7) is based, it is adaptable because all errors in the derivation can be nullified by the proper selection of the discharge coefficient, μ_0. As a consequence it has remained the foundation of all well known weir formulas.

Weisbach attempted to improve the formula by expressing μ_0 as a function of the depth of flow in the approach channel, or, what amounts to the same

thing, as a function of the mean velocity of approach. His formula (1844) is of the type:

$$\mu_0 = \mu \left\{ a + b \left(\frac{h_0}{p + h_0} \right)^2 \right\} \quad \text{.....................(8)}$$

in which, μ is the coefficient for a very high weir, and a and b are constants. Most of the weir formulas which have subsequently been proposed are of the Poleni-Dubuat-Weisbach form and differ essentially only in the values of μ, a and b.

The following formulas for μ_0 in terms of h_0 and p, in meters, are those most commonly used in Europe:

Bazin's formula (1888):

$$\mu_0 = \left(0.6075 + \frac{0.0045}{h_0} \right) \left\{ 1 + 0.55 \left(\frac{h_0}{h_0 + p} \right)^2 \right\} \quad \text{..........(9)}$$

Frese's formula (1890):

$$\mu_0 = \left(0.615 + \frac{0.0021}{h_0} \right) \left\{ 1 + 0.55 \left(\frac{h_0}{h_0 + p} \right)^2 \right\} \quad \text{.........(10)}$$

The formula of the Swiss Society of Engineers and Architects (1924):

$$\mu_0 = 0.615 \left(1 + \frac{1}{1\,000\, h_0 + 1.6} \right) \left\{ 1 + 0.5 \left(\frac{h_0}{h_0 + p} \right)^2 \right\} \quad \text{......(11)}$$

The writer's formula (1912):

$$\mu_0 = 0.605 + \frac{1}{1\,050\, h_0 - 3} + 0.08 \frac{h_0}{p} \quad \text{...............(12)}$$

The writer had tried in vain for many years to determine a reliable discharge formula of the Weisbach type which agreed with the results of experiments made in the Karlsruhe Hydraulic Laboratory, even on weirs with small values of p and h_0. Formula (12), which was finally proposed, differs from the type presented by Weisbach, but is in close agreement with the writer's tests.

For heads greater than 0.03 m., that is, for practically all cases which are met in practice, this formula can be simplified into:

$$\mu_0 = 0.605 + \frac{1}{1\,000\, h_0} + 0.08 \frac{h_0}{p} \quad \text{.................(13)}$$

The differences between the μ_0-values of Formulas (12) and (13) are approximately $\frac{1}{4}$ of 1% for $h_0 = 0.03$ m., while for $h = 0.04$ m., this discrepancy between the formulas is only about $\frac{1}{10}$ of 1% and becomes still smaller for higher heads.

Strictly speaking, Formulas (9) to (13) are not dimensionally correct. Some disturbance, which is not yet clearly understood, arises in the case of flow of water over sharp-crested weirs, which adds materially to the difficulties of determining a formula for the coefficient of discharge. In consequence of this disturbance the law of similarity cannot be strictly applied in the case of sharp-crested weirs because the coefficient, μ_0, is not exactly the same for any two sharp-crested weirs of different absolute heights, p, even if the ratios $\frac{h_0}{p}$, are identical.

The fact that the writer's discharge formula,

$$Q = \frac{2}{3}\left(0.605 + \frac{1}{1\,050\,h_0 - 3} + 0.08\,\frac{h_0}{p}\right)\sqrt{2\,g}\,b\,h_0^{\frac{3}{2}} \;\ldots\ldots(14)$$

determined from the results of experiments, gives for the metrical system values almost identical with those obtained from the formula,

$$Q = \frac{2}{3}\left(0.602 + 0.0832\,\frac{h}{p}\right)\sqrt{2\,g}\,b\,(h_0 + 0.00125)^{\frac{3}{2}} \;\ldots\ldots(15)$$

may lead to a possible explanation. A comparison of these two formulas indicates that the disturbance influences the discharge in such a way that the discharge is not directly proportional to the $\frac{3}{2}$ power of the head measured on the master gauge, but rather to that of an effective head which is about 1.25 mm. greater than that actually observed.

L. Prandtl has expressed the opinion that capillarity might be responsible for the form of the second term of Formula (12) and that the behavior may possibly be comparable to the rise of the water surface along a vertical wall, $h' = \sqrt{\frac{K}{\gamma}}$, in which, K is the coefficient of capillarity and γ, the density of the fluid.

Since h' is equal to 0.00267 m., Formula (12) may be changed accordingly to read:

$$\mu_0 = 0.605 + \frac{0.36}{h_0\sqrt{\frac{\gamma}{K}} - 1} + 0.08\,\frac{h_0}{p} \ldots\ldots\ldots\ldots\ldots(16)$$

The values obtained from Formula (16) are only about $\frac{1}{3}$ of 1% less than those obtained from Formula (12) for heads of 0.01 m., and the difference between the formula values does not exceed $\frac{1}{4}$ of 1% for heads greater than 0.015 m. These discrepancies are meaningless in the case of such low heads and practically disappear at higher heads.

Formula (15) may also be rewritten to take account of the capillarity coefficient, as follows:

$$Q = \frac{2}{3}\left(0.602 + 0.0832\,\frac{h_0}{p}\right)\sqrt{2\,g}\,b\left(h_0 + 0.47\sqrt{\frac{K}{\gamma}}\right)^{\frac{3}{2}} \ldots\ldots(17)$$

It is evident that Formulas (15), (16), and (17) are dimensionally correct, because $\sqrt{\frac{K}{\gamma}}$ is a length.

It is not yet definitely known, however, whether this disturbance is due to capillarity or to some other cause. In any event some phenomena exist which have the same effect on the discharge as a constant increase of head on the weir equal to $1\frac{1}{4}$ mm.

Formula (15) can be simplified and expressed in second-feet (with sufficient exactness) as follows:

$$Q = b\left(3.22 + 0.445\,\frac{h_0}{p}\right)(h_0 + 0.004)^{\frac{3}{2}} \text{ sec-ft} \ldots\ldots\ldots\ldots (18)$$

Formulas (9), (10), and (11) have been considered applicable with the following limitations:

(*a*) Formula (9):

$$0.1 < h_0 < 0.6 \text{ m.}$$

(*b*) Formula (10):

$$0.1 < h_0 < 0.6 \text{ m.}$$

(*c*) Formula (11):

$$0.025 < h_0 < 0.8 \text{ m.};\ p \geqq 0.3 \text{ m.};\ h_0 < p$$

Formulas (12) to (19), inclusive, are not necessarily restricted between any given limits. They apply in general to any sharp-crested vertical weir without end contractions as long as the jet is perfectly free, fully aerated, and not affected by back-water. Since a free overfall does not exist for heads less than about 0.01 m., this head is automatically the lower limit of applicability of the formulas. Obviously the accuracy of measurements with heads so small is not very great.

The formulas are applicable even to weirs of very low height, p, and have been found to agree with the tests of a weir only 0.056 m. high.

These formulas seem applicable, too, for all large heights and large heads in practical use, as long as it is possible to aerate the nappe fully. This may be deduced from the law of similarity. However, the writer advises against building sharp-crested weirs higher than 1.2 m. (4 ft.), for gauging purposes, and using such weirs for heads much greater than 0.6 m. (2 ft.).

If the bottom of the channel below the weir is at the same level as the bottom of the approach channel, the heads, h_0, on the weir should not be much greater than the height of the weir, p, because there is danger of the heads being affected by the back-water in the discharge channel. If the bottom of the discharge is lower than that of the approach, the formulas are in substantial agreement with the tests even if the heads are two to four times the weir height and perhaps still more. This may be seen clearly in Fig. 64.

The best agreement between the formulas and tests may be expected, however, if 0.1 m. $< p <$ 1.00 m. and 0.025 m. $< h_0 <$ 0.60 m. Weirs, therefore, should preferably be designed so that the heights and heads, under which they will be used, fall between these limits. In general, this can be attained through proper selection of the length of the weir crest. Weirs of ordinary spillway section with a cylindrical crest are to be preferred to sharp-crested weirs if very large rates of discharge are to be measured, for if the discharge over a sharp-crested weir becomes very large, it is difficult to provide adequate ventilation of the jet.

The tests of the Swiss Government Department of Hydraulics have shown that weir measurements for heads of 0.8 m. are still reliable and Fig. 65 shows

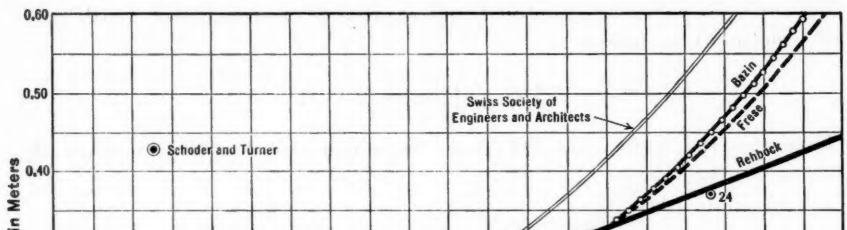

FIG 64.—μ_0-VALUES FOR WEIRS 0.5 FEET HIGH.

that the writer's formulas give values differing from these measurements by only fractional parts of 1% up to heads of 0.8 m.

Comparison of Formulas in Use in Europe with Recent Tests.—The writer has restricted himself to a comparison of the four formulas for the flow of water over sharp-crested weirs, which are in common use in Europe

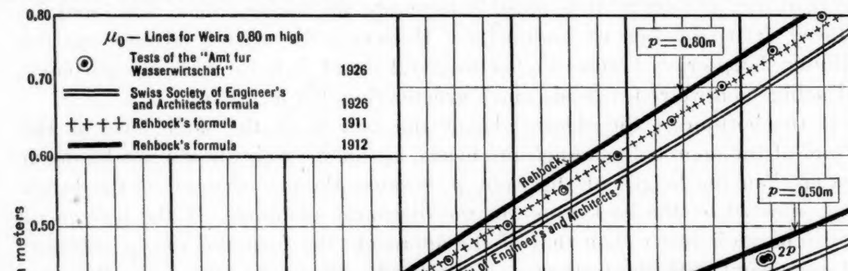

FIG. 65.—COMPARISON OF THE μ_0-VALUES.

(Formulas (9), (10), (11) and (12)). A comparison of the formulas of Francis, Fteley and Stearns, and H. W. King, M. Am. Soc. C. E., which are commonly used in the United States, has not been made.

Relatively few new tests have come to the writer's attention in recent years. Most of them were for only one height of weir and, therefore, did not permit a thorough comparison. Since the publication of the writer's formulas, the only European tests which have come to his knowledge with enough information, so that the μ_0-values could be calculated from the data are those of Fr. Schaffernak, in Vienna, Austria (1915), Erik G. W. Lindquist, M. Am. Soc. C. E., in Stockholm, Sweden (1926), and the Swiss Government Department of Hydraulics (1926). All these gaugings were made with the greatest care, but with different equipment and different rating methods.

Schaffernak published* the results of seventeen measurements for a weir 1 m. long and 0.56 m. high for mean heads lying between 0.0288 m. and 0.3082 m. In these tests only one ventilator pipe with a cross-sectional area of about 5 sq. cm. was provided for aerating the space under the jet. This pipe certainly could not have provided sufficient air for heads greater than 0.2 m. Therefore, only the data for heads less than 0.2 m. may be considered to be correct. The results of these eight tests are shown in Table 63.

TABLE 63.—SCHAFFERNAK'S EXPERIMENTS ON A WEIR 0.56 METER HIGH.

No.	h_0, in meters.	μ_0', according to tests of Schaffernak.	μ_0, according to Formula (12) of Rehbock.	Difference, $\Delta \mu_0 = \mu_0' - \mu_0$.	Difference, $\Delta \mu_0$ percentage.
1	0.0288	0.6444	0.6458	−0.0014	−0.22
2	0.0488	0.6313	0.6327	−0.0014	−0.22
3	0.0878	0.6301	0.6308	−0.0007	−0.11
4	0.1427	0.6295	0.6322	−0.0027	−0.43
5	0.1583	0.6330	0.6337	−0.0007	−0.11
6	0.1693	0.6354	0.6349	+0.0007	+0.11
7	0.1777	0.6360	0.6358	+0.0002	+0.03
8	0.1890	0.6374	0.6371	+0.0003	+0.05
Arithmetic mean	......	......	......	− 0.0007	−0.11
Mean discrepancy	......	......	......	0.0010	0.16

The maximum difference between the μ_0-values calculated from Formula (12) and those observed by Schaffernak for heads between 0.0288 m. and 0.187 m., is only 0.0027, or 0.43 per cent. The mean absolute percentage differences of observed and computed values, according to Formula (12), is 0.16.

Lindquist's experiments,† in Stockholm, were made on a weir, 0.50 m. in height, for heads that ranged between 0.124 m. and 0.452 m. (Table 64).

The maximum difference between observed μ_0-values and those of the writer's formula is 0.0042, or 0.64 per cent. The mean of all percentage

* Mitteilungen der Versuchsanstalt für Wasserbau über ausgeführte Versuche, Wien, 1915, 1, Folge B, "Die Eichung eines scharfkantigen belüfteten Ueberfallwehres ohne seitliche Strahleinzwängung (Rehbock'scher Ueberfall)".

† Sent to the writer by letter of April 3, 1926.

differences is 0.28. The results of further proposed tests of Lindquist for other heights of weirs are not yet available.

TABLE 64.—LINDQUIST'S EXPERIMENTS ON A WEIR 0.50 METER HIGH.

No.	h_0, in meters.	μ_0', according to tests of Lindquist.	μ_0, according to Formula (12) of Rehbock.	Difference, $\Delta\mu_0 = \mu_0' - \mu_0$.	Difference, $\Delta\mu_0$, percentage.
1	0.1242	0.6335	0.6327	+0.0008	+0.13
2	0.1400	0.6357	0.6343	+0.0014	+0.22
3	0.1972	0.6403	0.6415	−0.0012	−0.19
4	0.2685	0.6475	0.6516	−0.0041	−0.63
5	0.3165	0.6545	0.6587	−0.0042	−0.64
6	0.3686	0.6668	0.6666	+0.0002	+0.03
7	0.3977	0.6714	0.6710	+0.0004	+0.06
8	0.3983	0.6715	0.6711	+0.0004	+0.06
9	0.4508	0.6820	0.6793	+0.0027	+0.40
10	0.4517	0.6825	0.6794	+0.0031	+0.46
Arithmetic mean.....	......	......	......	−0.00005	−0.01
Mean discrepancy.....	......	......	......	0.00185	0.28

The measurements of the Swiss Department of Hydraulics, in Berne,* agree equally well with the results of Formula (12). These measurements were made on a sharp-crested weir, 0.8 m. high, at the Amsteg Power Plant, in Switzerland, and the data were given to the Swiss Society of Engineers and Architects to be used in preparing standard specifications for weir measurements.

Table 65 shows the mean values of μ_0, which are presented in Table 26 of the original report of the Swiss Department of Hydraulics previously mentioned, and the corresponding μ_0-values according to Formula (12).

The mean discrepancy between observed and computed values of μ_0 is 0.25 per cent. The coefficients of discharge reported by the Swiss Department of Hydraulics agree still more closely with the values of μ_0 computed by the writer's earlier formula (1911)†:

$$\mu_0 = 0.605 + \frac{1}{1\,100\,h_0} + \frac{h_0}{12\,p} \qquad (19)$$

The greatest discrepancy between the observations and the results of Formula (19) do not in any case exceed 0.0024, and the mean discrepancy between all observed and computed values is only 0.0009, or 0.14 per cent. A comparison of the μ_0-values of these observations and those of Lindquist with the writer's μ_0-lines for the same heights of weir, is shown in Fig. 65.

A comparison of the coefficients of discharge determined experimentally in Vienna, Stockholm, and Amsteg for different testing layouts, with the corresponding coefficients calculated by Formula (12) shows a maximum discrepancy of 0.64 per cent. The mean discrepancy for the entire thirty-three measurements is only 0.24 per cent.

* Mitteilungen des Eidgenössischen Amtes für Wasserwirtschaft, herausgegeben unter Leitung von Dr. Mutzner, 1926, No. 18.

† *Zeitschrift* des Verbandes deutscher Architekten-und Ingenieur Vereine, 1, Jahrgang No. 1, 6 Januar, 1912.

Referring to the form of the μ_0-lines in Fig. 65, it is noteworthy that the writer in setting up Formulas (12) and (19), took into consideration only observations for weirs 0.5 m. or less in height and for heads up to 0.18 m. in magnitude for the latter and up to 0.231 m. for the former.

TABLE 65.—EXPERIMENTS AT AMSTEG BY THE SWISS DEPARTMENT OF HYDRAULICS.

No.	h_0, in meters.	μ_0' according to tests of Amt. für Wasser-wirtschaft.	μ_0, according to Formula (12) of Rehbock.	Difference, $\mu_0 = \mu_0' - \mu_0$.	Difference, μ_0, in percentage.	μ_0, according to Formula (17) of Rehbock.	Difference, $\mu_0 = \mu_0' - \mu_0$.	Difference, μ_0, in percentage.
1	0.1	0.6262	0.6248	+0.0014	+0.22	0.6252	+0.0010	+0.16
2	0.15	0.6269	0.6265	+0.0004	+0.06	0.6271	−0.0002	−0.03
3	0.2	0.6294	0.6298	−0.0004	−0.07	0.6307	−0.0013	−0.21
4	0.25	0.6323	0.6338	−0.0015	−0.24	0.6347	−0.0024	−0.38
5	0.3	0.6376	0.6382	−0.0006	−0.10	0.6295	−0.0019	−0.30
6	0.35	0.6428	0.6427	+0.0001	+0.01	0.6442	−0.0014	−0.22
7	0.4	0.6489	0.6474	+0.0015	+0.23	0.6491	−0.0002	−0.03
8	0.45	0.6542	0.6521	+0.0021	+0.33	0.6541	+0.0001	+0.02
9	0.5	0.6596	0.6569	+0.0027	+0.40	0.6590	+0.0006	+0.09
10	0.55	0.6646	0.6617	+0.0029	+0.43	0.6640	+0.0006	+0.09
11	0.6	0.6694	0.6666	+0.0028	+0.41	0.6691	+0.0003	+0.05
12	0.65	0.6741	0.6715	+0.0026	+0.39	0.6742	−0.0001	−0.02
13	0.7	0.6737	0.6764	+0.0023	+0.34	0.6793	−0.0006	−0.09
14	0.75	0.6832	0.6813	+0.0019	+0.29	0.6844	−0.0012	−0.18
15	0.8	0.6877	0.6862	+0.0015	+0.22	0.6895	−0.0018	−0.26
Arithmetic mean....	....	......	......	+0.0013	+0.20	......	−0.0006	−0.09
Mean discrepancy....	....	......	......	0.0016	0.25	......	0.0009	0.14

Even if the μ_0-values for larger weir and overflow heights were obtained only by consideration of the law of similitude as a basis, they show a variation of less than ¼ of 1% from the μ_0-values later obtained by the Swiss Department of Hydraulics on weir and overflow heights of 0.80 m.

Experiments were also conducted in the Hydraulic Laboratory at Dresden for the purpose of checking Formula (12), but the details of these tests were not published. A letter from H. Engels, Director of this Laboratory, stated, however, that the agreement of the formula with the experimental results was most satisfactory.

M. Carstanjen* shows discharge curves for two series of experiments made by the late A. Koch in Darmstadt and the corresponding discharge curves according to Formula (12). The experimental and calculated curves lie near together, but the data are not presented in sufficient detail, so that a numerical comparison of the μ_0-values can be made.

In addition to these more recent experimental verifications in the various hydraulic laboratories of Europe, the writer has also a report from Dr. Hahn, of the J. M. Voith Company, in Heidenheim, Germany, that Formula (12) agreed very well with experimental results obtained in the Voith Turbine Laboratory.

* Koch-Carstanjen, "Von der Bewegung des Wassers und den dabei auftretenden Kräften", Berlin, 1926, Julius Springer Verlag, p. 156.

The extensive gaugings of the flow over sharp-crested weirs made in the last years by the Swiss Society of Engineers and Architects have not yet been published and, therefore, could not be included in the comparisons which are presented herewith.

The experiments* by Floyd A. Nagler, M. Am. Soc. C. E., for a weir of 1.124 m. high, gives μ_0-values still higher than those of Bazin and from 3.6 to 4.9% higher than those of Formula (12). These results show, that the hydro-chemical method is not a reliable way to verify weir measurements.

Weir Gaugings by Schoder and Turner.—The authors' paper, presenting the results of 2 438 gaugings, is the most comprehensive compilation of the records of gauging of sharp-crested suppressed weirs that has come to the writer's attention. It includes the results of experiments on weirs of different heights, under wide ranges of heads, and is a source of information of inestimable value to the hydraulic experimenter.

After a detailed study of the 1 017 measurements in Series D to P (Table 1†), the writer has come to the conclusion that these measurements for weirs of normal height (0.5 ft. to 4 ft.) under normal heads between 1 in. and 2 ft. were made with a particularly high degree of accuracy. For heads less than 1 in. (0.025 m.), the data are somewhat erratic, but this is considered unavoidable because measurements of heads could only be made by means of the float-gauge without a vernier scale and, therefore, could not reach a very great precision.

The accuracy of the data for the weirs, 5.5 and 7.5 ft. in height is open to some question. If the coefficients of discharge, μ_0, are plotted as abscissas and the corresponding heads, h_0, as ordinates for the weirs of different heights, p, it will be seen that the coefficient of discharge for a given head decreases regularly as a function of the height of weir for weirs which are 4 ft. or less in height. However, some of the coefficients of discharge for weirs 5.5 and 7.5 ft. high are partly greater than those for the weir 4 ft. high under the same heads. It is wholly unreasonable to think that, for a given head, h_0, the coefficient of discharge becomes greater when the weir height is increased because the velocity of approach must become increasingly slower.

The erratic behavior of the data for the weirs, 5.5 and 7.5 ft. in height, is at once traceable to the construction of the approach channel shown in Fig. 6.‡ The surface of the fixed bottom of the approach channel over the 30-in. supply pipe appears to be only about 5 ft. below the weir crest, while the movable bottom near the weir lies below this level during the tests of the two highest weirs. Undoubtedly, the disturbance arising from this type of construction changed the flow in such a way that the "effective" height of the weir was actually much less than the height measured from the crest to the bottom of the approach channel near the weir. The fact that the data for the 5.5 and 7.5-ft. weirs (Series D, K, L, M, and O, Table 1) are obviously unreliable is, however, of little importance, because sharp-crested weirs more than 4 ft. in height are almost never used in practice.

* "Verification of the Bazin Weir Formula by Hydro-Chemical Gaugings," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 105.

† *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, pp. 1396–1397.

‡ *Loc. cit.*, p. 1407.

The writer also feels that the accuracy of the data for heads greater than 0.5 to 0.6 m. (1⅔ to 2 ft., and more) is also questionable. There are disturbances in the law of similarity at low heads, but these are barely perceptible if the head on the weir is about 0.3 m. (1 ft.) and they disappear almost entirely at higher heads. If the μ_0-values of Schoder and Turner for high heads were correct some new disturbance must occur which makes the coefficients of discharge increase suddenly if the head is greater than about 2 ft. This is hardly to be expected and is not borne out by the experiments of the Swiss Government Department of Hydraulics which have been summarized in Table 65 and Fig. 65.

The most common cause of such abrupt changes in the discharge coefficient-head relation at the higher heads is insufficient ventilation of the space between the jet and the weir plate. Whether the sudden divergence of the authors' measurements from the $\mu_0\,h$-curve, which is nearly straight, is chargeable to insufficient ventilation, or to some other cause, cannot be stated definitely without having at hand more detailed information concerning the arrangement of the testing layout.

If the measurements for heads greater than 0.6 m. (2 ft.) are neglected and the formulas are applied only to heads less than 2 ft., as Bazin and Frese have recommended, the comparison still covers nearly the entire practical range of heads, which is generally used for sharp-crested weirs.

The coefficient of discharge, μ_0, in Formula (7) was calculated for each of the Schoder and Turner gaugings for all heads between 0.01 m. and 0.60 m. These μ_0-values are compared with the corresponding μ_0-values determined by Formulas (9) to (12), inclusive. Figs. 66 to 71 show graphically the comparison of the observations for weirs ¾, 1, 1½, 2, 3, and 4 ft. high, respectively, as found by Formulas (9), (10), (11), and (12). (See, also, Fig. 64.) The limits of the formulas, as given by their respective authors, are shown by black points. Beyond these limits the curve of the formula is shown only by lighter lines.

These diagrams show that, in all cases, Formula (9) gives much higher values of discharge for a given head than the Schoder and Turner tests indicate. Formula (10) shows a somewhat better agreement, but all the values are still too high. Formula (12) shows the best agreement with the results of the authors' experiments. Formula (11) also agrees quite well with the experimental results, but shows mean differences nearly twice as large as those from the writer's Formula (12).

Fig. 72 shows the mean differences between the coefficients of discharge, μ_0, determined from the results of the authors' tests and the corresponding μ_0-values calculated by Formulas (9), (10), (11) and (12) for all heads between 0.025 and 0.60 m. and for $h_0 < 2\,p$.

Fig. 72 shows that the formulas of Bazin and Frese give values of μ_0 which are on the average approximately 6.3 and 3.2% greater than the corresponding μ_0-values determined from the experiments of Schoder and Turner. Formula (11) shows a mean discrepancy of 0.70% from the authors' tests, while in the case of Formula (12), this discrepancy is 0.38 per cent.

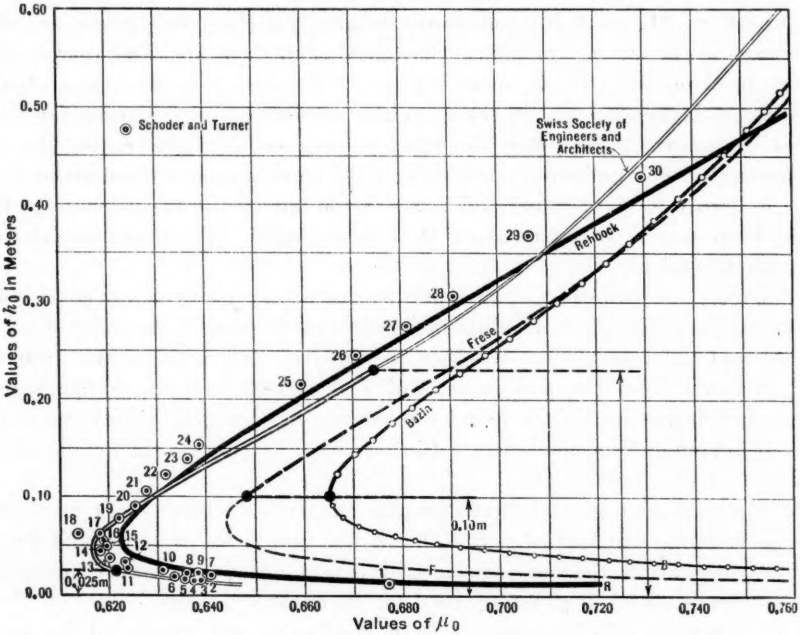

FIG. 66.—μ_0-VALUES FOR WEIRS 0.75 FEET HIGH.

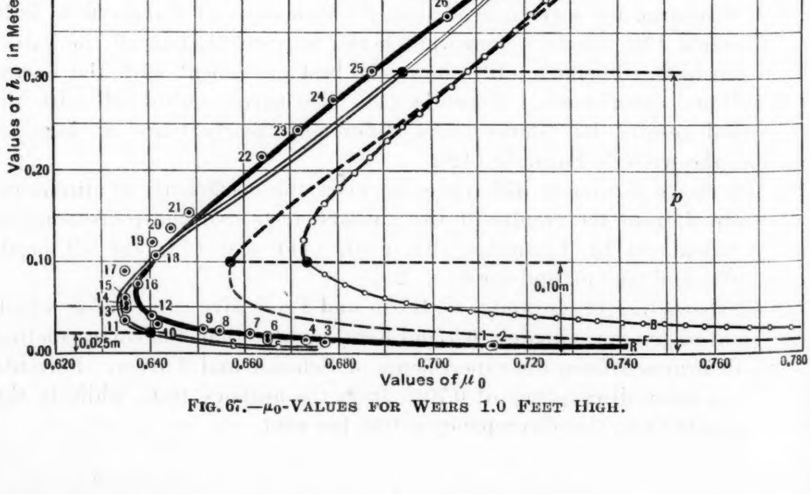

FIG. 67.—μ_0-VALUES FOR WEIRS 1.0 FEET HIGH.

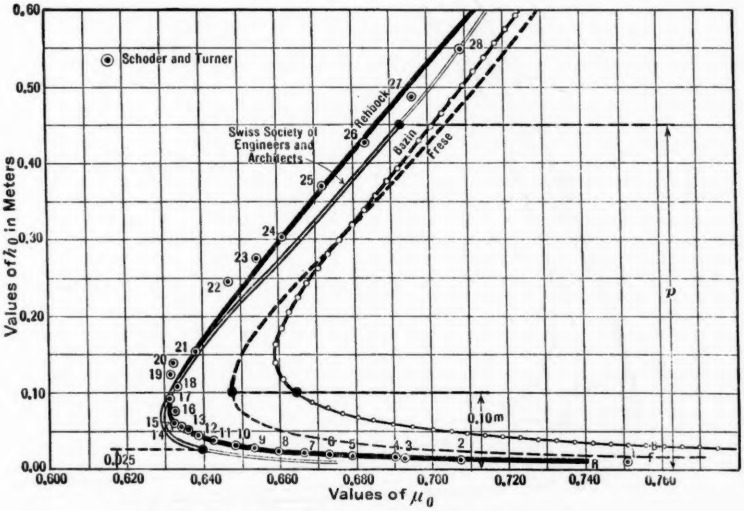

FIG. 68.—μ_0-VALUES FOR WEIRS 1.5 FEET HIGH.

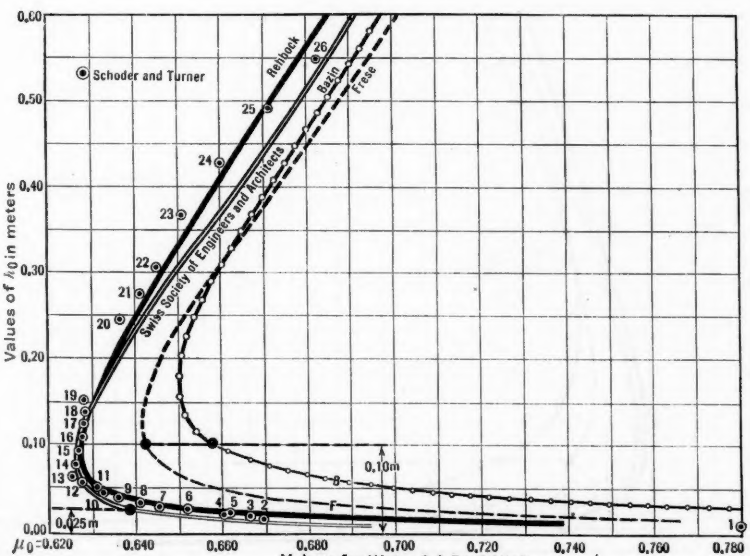

FIG. 69.—μ_0-VALUES FOR WEIRS 2.0 FEET HIGH.

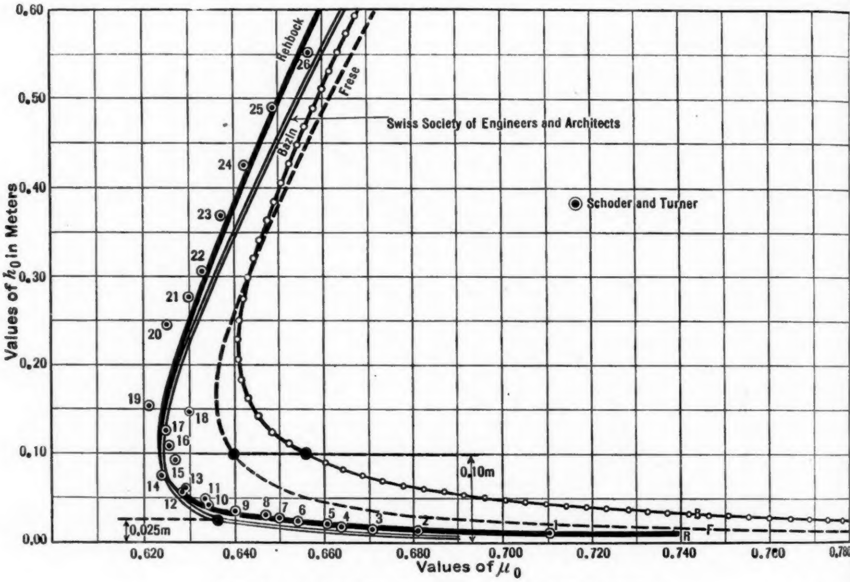

FIG. 70.—μ_0-VALUES FOR WEIRS 3.0 FEET HIGH.

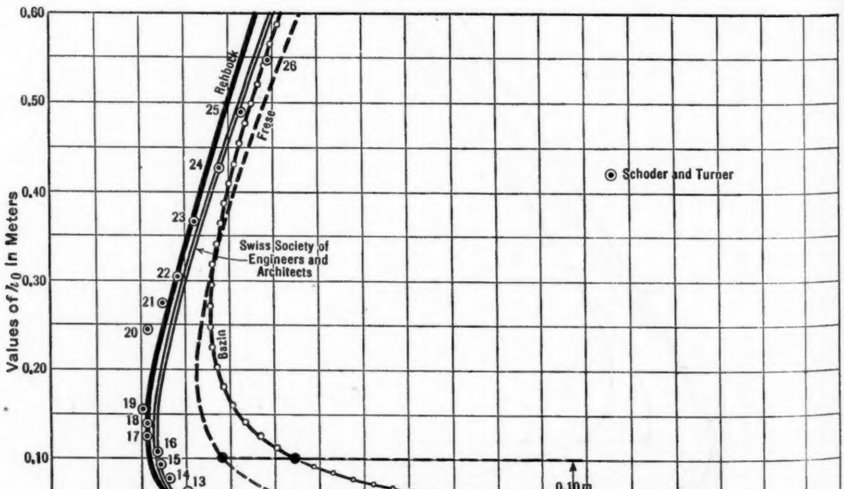

FIG. 71.—μ_0-VALUES FOR WEIRS 4.0 FEET HIGH.

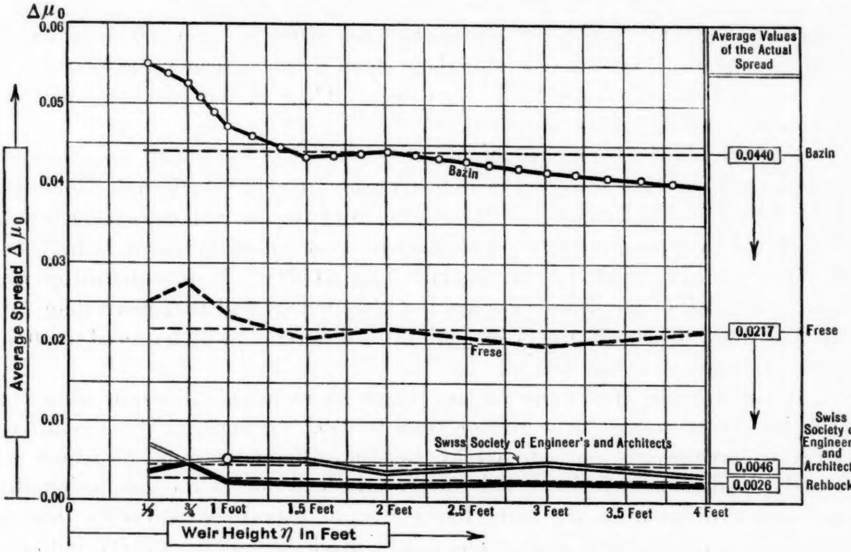

FIG. 72.—COMPARISON OF AVERAGE VALUES OF $\Delta\mu_0$.

It should be stated that the discrepancies mentioned between the several formulas and the observations of Schoder and Turner are the mean values of the absolute percentage differences based on the value of μ_0 according to the particular formula under consideration. In other words the figures represent the arithmetic mean of the percentage deviations, without regard to algebraic sign, between each of the discharge coefficients given by the authors and the corresponding discharge coefficients according to the respective formulas.

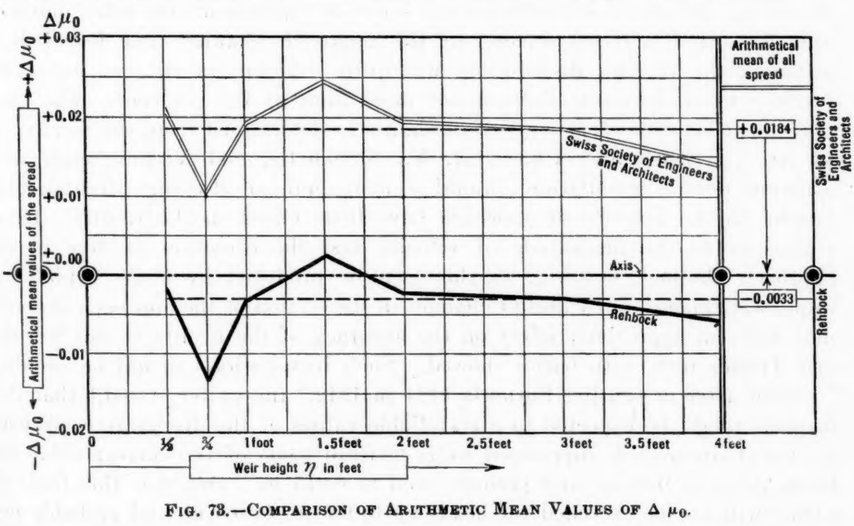

FIG. 73.—COMPARISON OF ARITHMETIC MEAN VALUES OF $\Delta\mu_0$.

Fig. 73 shows the average absolute values of the differences between the observations of Schoder and Turner and the corresponding μ_0-values of

Formula (12) for very small heads between 0.015 m. and 0.025 m. on the different weirs. The difference for these small heads of nearly the same height is the arithmetic average of the μ_0-values. These differences are small and reach only 0.0023, or 0.33 per cent.

Fig. 74(a) shows the differences between the coefficients of discharge determined from the test data of Schoder and Turner, Schaffernak, Lindquist, and the Swiss Department of Hydraulics and the corresponding coefficients calculated by Formula (12). The magnitude of the differences is indicated by the symbols shown in the legend (Fig. 74(d)). It is noteworthy that nearly 50% of all the differences are less than 0.0025 and that more than 75% are less than 0.0050. With one exception the differences which exceed 0.010 are only for heads less than 0.02 m.

It will be seen that there are no larger zones in this diagram where the differences are consistently greater than 0.0050. In general, these relatively large differences are surrounded by smaller differences some of which are positive and some negative. This fact would lead one to the conclusion that the larger differences are more likely due to experimental errors than to inexactness of Formula (12). The heavy black line (Fig. 74(a)) indicates the field usually covered by weir measurements.

From Fig. 74(b) it is possible to determine the effect of an error of 0.0001 m. in measuring the value of the head, h_0, on the corresponding value of μ_0.

Slight irregularities in the magnitude of the velocity of approach to the weir as they occur in a normal channel do not have a material influence on the coefficient of discharge as long as the cross-section of the approach channel is the same throughout its entire length and as long as the flow is in an axial direction. Schaffernak conducted two series of experiments on weirs, one with and the other without screens in the approach channel and found that, although the velocity distribution was quite different in the two cases, the coefficients of discharge differed not more than by 0.2 per cent. The close agreement between coefficients of discharge as determined from the tests made in Amsteg, Karlsruhe, Ithaca, N. Y., Stockholm, and Vienna, each with different testing installations, would seem to indicate also that slight irregularities in the velocity of approach have little effect on the results. Large variations in the magnitude of velocity and the direction of flow in the approach channel, however, as they can be produced by baffles which are improperly placed, or by abrupt changes in the section of the approach channel, may have an appreciable effect on the accuracy of the results as the Schoder and Turner tests with baffles showed. Such installations should be avoided.

Soon after presenting Formula (12) in 1912,* the writer stated,† that this formula might be expected to give reliable values of the discharge coefficient, μ_0, for sharp-crested, suppressed weirs "within 0.005 of the correct value for heads down to 0.02 m. and perhaps even to 0.015 m.", and that this limit of 0.005 "will not be exceeded for heads up to 0.5 m. to 0.6 m. and probably not

* Handbuch der Ingenieurwissenschaften, dritter Teil, 2. Band, 1. Abteilung, Leipzig 1912, p. 58.

† *Zeitschrift* des Verbandes Deutscher Arch. u. Ing. Vereine, 1913, Nr. 1.

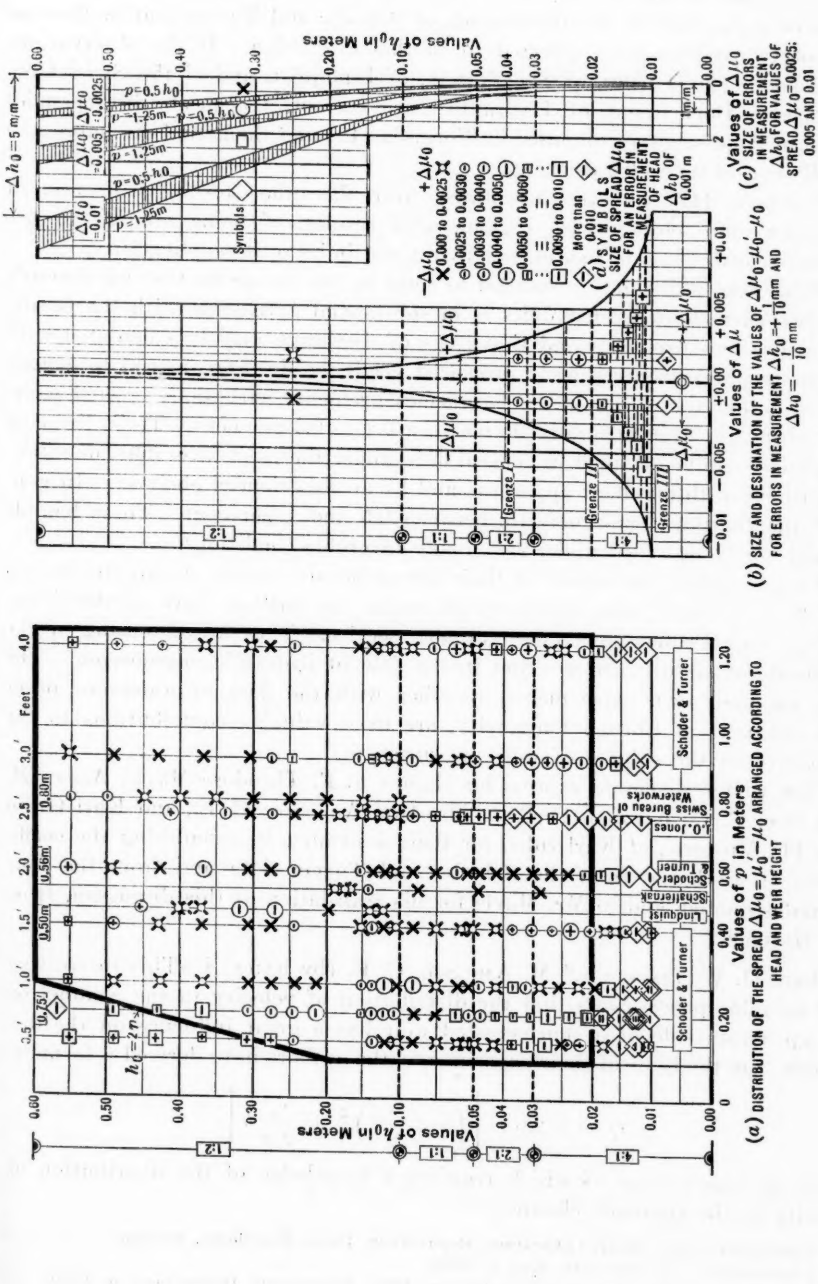

Fig. 74.—Comparison of the μ_0'-Values.

even for larger heads if the nappe is beyond question fully ventilated and not affected by back-water."

The expectations of the writer thus expressed in January, 1913, seem to have been justified by the experiments of Schoder and Turner and by those of the other experimenters referred to in this discussion. If the observations of Schoder and Turner, of Schaffernak, of Lindquist, and of the Swiss Governmental Department of Hydraulics are correct, the mean discrepancies between the discharge computed by Formula (12) and the actual discharge will hardly exceed 0.2 to 0.3 per cent.

Formula (12) has the advantage, over the other formulas in use, of agreeing more closely with experimental results, of being simpler to use mathematically, and of having practically no limits to its applicability.

While the writer has attempted to show in the foregoing that his formula for flow over sharp-crested weirs is in substantial agreement with the results of precise measurements and that rates of discharge may be calculated with ample accuracy from properly measured heads, he does not intend to convey the idea that the errors in discharge measurements with sharp-crested weirs will always or even generally lie between 0.2 to 0.3 per cent. These limiting accuracies can be attained in general only under the most favorable laboratory conditions, with excellent apparatus and by an experienced observer. In general, the limits of error may lie between 0.5 and 1 per cent. These bounds should not be exceeded under reasonably favorable conditions.

In presenting the results of their comprehensive series of experiments on the flow of water over sharp-crested weirs the authors have rendered the Engineering Profession a most valuable service. They are deserving of the gratitude of all who are working in the field of hydraulic engineering. The data on their own experiments, together with the data of numerous other experimenters at Cornell University, are most valuable contributions to the literature on the subject of weir measurements.

The writer wishes to express his thanks to F. Theodore Mavis, Assoc. M. Am. Soc. C. E., Kenneth C. Reynolds, Jun. Am. Soc. C. E., and Karl Grein and Th. Musterle, of Karlsruhe, for their assistance in calculating the coefficients of discharge from the Schoder and Turner data and from the four formulas quoted, and to Mr. Mavis for his translation of this discussion from the German.

ERIK G. W. LINDQUIST,* M. AM. SOC. C. E. (by letter).†—This interesting and valuable paper shows that the distribution of velocity in the channel on the up-stream side of a sharp-crested weir has a great influence on the discharge. On the basis of their experiments, the authors have derived a formula,

$$Q = 3.33\ L \left[\left(h + \frac{v_a^2}{2\,g} \right)^2 + \frac{v_b^2}{2\,g}\, h \right]$$

(D),‡ the application of which requires a knowledge of the distribution of velocity in the approach channel.

* Research Engr., Hydr. Laboratory, Royal Tech. Univ., Stockholm, Sweden.

† Received by the Secretary, May 4, 1928.

‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1399.

This formula does not seem to have a correct theoretical basis. The first term in the parenthesis is of the dimension, $L^{\frac{3}{2}}$, while the second has the dimension, L^2, and is not "of the nature of a volume", since the coefficient, 3.33, has the dimension, $L^{\frac{1}{2}}$. Such dimensionally incorrect formulas should be avoided as they can only express the result obtained with a particular experimental arrangement, which usually cannot be duplicated by other experimenters. In spite of the fact that, in most cases, the Schoder and Turner tests show a very irregular distribution of velocity, a theoretical treatment of them shows to some extent the influence of the factors involved.

Consider, first, a perfect fluid (frictionless and not affected by molecular forces, such as surface tension) under the influence of gravity and flowing over an infinitely high, vertical weir, with a horizontal and sharp crest. Since no other forces are considered as acting, the stream lines are parallel and have the same energy content. The factors determining the discharge are the acceleration of gravity and the head above the crest.

The ratio, $\frac{Q}{L}$, can be found, with the aid of dimensional analysis, to be,

$$\frac{Q}{L} = \phi \sqrt{2\,g}\, h^{\frac{3}{2}} \qquad (20)$$

Combining this with the formula,

$$Q = \frac{2}{3}\,\mu\, L \sqrt{2\,g}\, h^{\frac{3}{2}} \qquad (21)$$

which is derived by the usual method of integrating the elementary discharges of the sheet, the value of ϕ is found to be,

$$\phi = \frac{2}{3}\,\mu \qquad (22)$$

For a sharp crest, the coefficient, ϕ, can be shown to be mathematically equal to 0.407. The discharge coefficient, μ, is thus 0.611.

Under these assumptions, μ has a constant value for all heads. If the weir is not infinitely high, μ can no longer be constant, but must vary with the height of the weir and the head. Indicating the height of the weir as w, the head as h, and the velocity head as k, μ must be a function of the ratios, $\frac{h}{w}$, $\frac{k}{h}$, or $\frac{h}{h + w}$. Theoretically, μ must be a constant when $w = \infty$, and equal to 1 when $h = \infty$, or when $w = 0$. From these considerations it is evident that it is not sufficient to assume that,

$$\mu = m + n\,\frac{h}{w} \qquad (23)$$

since μ would then become ∞ when the height of the weir is very small in comparison with the head. The expression, $\frac{h}{h + w}$, satisfies the theoretical conditions.

For a perfect fluid, the velocity head is,

$$k = \frac{v^2}{2\,g} = \frac{Q^2}{L^2\,(h + w)^2\,2\,g} \qquad (24)$$

and combining Equation (24) with Equations (20) and (21), the resulting ratio is,

$$\frac{k}{h} = \phi^2 \left(\frac{h}{h + w}\right)^2 = \frac{4}{9}\,\mu^2 \left(\frac{h}{h + w}\right)^2 \qquad (25)$$

Therefore,

$$\mu = f_1 \left(\frac{k}{h}\right) \qquad (26)$$

Considering now a real fluid (viscous, adhesive, and having a certain surface tension) flowing over a weir of finite height, the energy content of the different stream lines is not the same, due to the viscosity. For this reason, two weirs having the same value of $\frac{h}{h + w}$ will show the same value of μ only when the distribution of velocity is the same at the sections at which the head is measured. If these distributions are determined by the viscosity alone, they can be similar only when the values of the Reynolds' number are the same. This criterion of similarity is the product of the mean velocity in the section, a length characterizing the section, and the reciprocal of the kinematic viscosity.

Thus,

$$E = \frac{v\,1}{\nu} \qquad (27)$$

The length can be chosen as the hydraulic radius, $R = \frac{A}{P}$, giving,

$$E = \frac{1}{\nu}\,\frac{Q}{A}\,\frac{A}{P} = \frac{1}{\nu}\,\frac{Q}{P} \qquad (28)$$

In this case, P, the wetted perimeter, is, $L + 2\,(h + w)$. Hence,

$$E = \frac{1}{\nu}\,\frac{Q}{L + 2\,(h + w)} = \frac{1}{\nu}\,\frac{Q}{L}\,\frac{1}{1 + 2\left(\frac{h + w}{L}\right)}$$

Among other things, the Reynolds' number is dependent on the width and height of the weir and the operating head. Consequently, μ must be a function of E and the complete expression is

$$\mu = F\left\{\frac{k}{h}, E\right\} \qquad (29)$$

Experiments to determine the loss of head in smooth pipes and the resistance of thin smooth plates, towed through still water, have shown that the Reynolds' number enters the resulting equation as $f\left(\frac{1}{E}\right)$ rather than as $f\,(E)$, showing that at high speeds or large dimensions the viscosity does not affect the magnitude of these quantities. Hence, for practical cases, where L and w

are usually large, the influence of the viscosity can be neglected. However, it must be stated that this conclusion only holds true when the type of flow is turbulent. For very low heads on high weirs, laminar flow may occur, causing a change in the conditions.

In most cases met in practice, the distribution of velocity in the cross-section at which the head is measured, is determined chiefly by other forces or arrangements than the viscosity and is not governed by any known mechanical law. To take into account the influence of the distribution of velocity on the discharge, the relation between the velocity heads as computed from the mean velocity may be compared with the mean kinetic energy of the individual stream lines or mass elements.

Considering a mass element, $d\,m$, flowing through a cross-sectional area, $d\,A$, with a velocity, v_y,

$$d\,m = \rho\,d\,A\,v_y, \qquad \text{(30)}$$

the velocity head is,

$$d\,k = \frac{\rho\,d\,A\,v_y\,v_y^2}{2\,g} \qquad \text{(31)}$$

and for the whole cross-section, the kinetic energy is,

$$\frac{\rho}{2\,g}\int_0^A v_y^3\,d\,A \qquad \text{(32)}$$

The mean velocity at the section is,

$$v_m = \frac{\int_0^A v_y\,d\,A}{A} \qquad \text{(33)}$$

and the total mass flowing through the section is,

$$m = \rho\int_0^A v_y\,d\,A \qquad \text{(34)}$$

Hence, the mean velocity head is,

$$k_m = \frac{\rho}{2\,g}\,\frac{\left[\int_0^A v_y\,d\,A\right]^3}{A^2} \qquad \text{(35)}$$

The ratio between these differently computed velocity heads is,

$$\delta = \frac{A^2\int_0^A v_y^3\,d\,A}{\left[\int_0^A v_y\,d\,A\right]^3} \qquad \text{(36)}$$

As δ is not a constant, but varies from case to case, a further condition arises, namely, that,

$$\mu = \phi\left\{\delta\right\} \qquad \text{(37)}$$

Neglecting the influence of purely viscous flow, the relation thus far discovered is,

$$\mu = \psi\left[\frac{k}{h}, \delta\right] \qquad \text{(38)}$$

When the distribution of velocity is determined by the viscosity alone, δ must be a function of E.

In the Schoder and Turner experiments, the value of δ, computed on the basis of the curves for the vertical element at mid-stream, varies as follows:

Series E, Run 53 to 57	1.07
" I, Run 54 to 58	1.08
" I, Fence 1	1.60
" I, Fence 3	2.08
" D, Run 101 to 105	1.80
" D, L, M (Fig. 46)*	2.0

However, the question arises as to whether δ completely determines the influence of the velocity distribution on μ. Imagine, for instance, two distribution curves with the same value of δ and with the same mean velocity, but with different surface and bottom velocities, as shown in Fig. 75. It seems possible that the μ-values will differ. In order to find other factors determining the μ, consider another problem capable of being treated mathematically, namely, the discharge of a fluid through a very long rectangular orifice in the bottom of an infinitely high basin (Fig. 76).

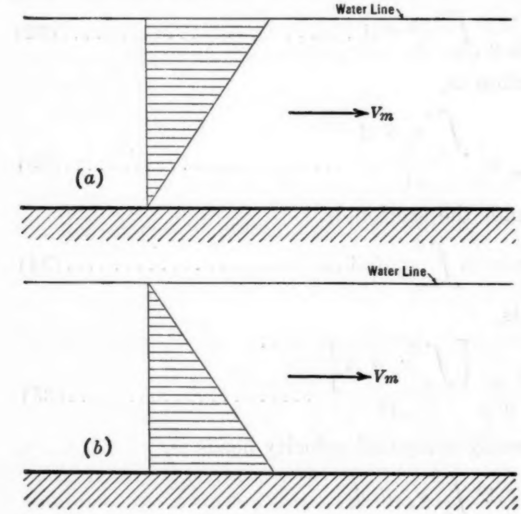

FIG. 75.

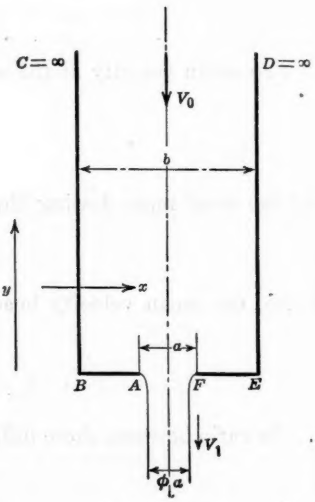

FIG. 76.

Assume that the flow is the result of an external pressure, p_0, on the surface instead of the force of gravity. Under the assumption that viscosity is not affecting the flow, the stream lines are parallel at infinity and have the same energy content.

From Bernouilli's theorem,

$$p_0 + \rho \frac{v_0^2}{2} = p + \rho \frac{v^2}{2} \qquad (39)$$

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1459.

in which, p and v are the pressure and velocity at any point and ρ is the density of the fluid. At some point beyond the orifice, the stream lines are parallel and have the same velocity, v_1, which the outer stream lines have had since leaving the orifice, because the surface of the jet is under constant pressure. Hence,

$$p_0 + \rho \frac{v_0^2}{2} = \rho \frac{v_1^2}{2} \qquad (40)$$

If q is the discharge per unit length and ϕ is the coefficient of contraction (see Fig. 76),

$$q = v_0 b = \phi a v_1 \qquad (41)$$

In order to find an expression for computing ϕ apply the law according to which the change in momentum between the initial and final sections is equal to the forces acting. The change in momentum is $\rho q (v_1 - v_0)$, and the forces are due to the pressures, p_0, and the pressures on the walls. These latter pressures may be denoted by $\int p\, d x$ and $\int p\, d y$. The integration is to be made over the entire boundaries and the differentials have the following values:

	$d x$.	$d y$.
Stretch A-B	—	0
" B-C	0	+
" D-E	0	—
" E-F	—	0

Hence from symmetry,

$$\rho q (v_1 - v_0) = p_0 b + \int_A^F p\, d x \qquad (42)$$

Combining Equation (42) with Equations (39), (40), and (41), and simplifying, the result is,

$$\phi = \frac{1}{1 + \sqrt{1 - \frac{a}{b} + \frac{a}{q^2}\int_A^F v^2\, d x}} \qquad (43)$$

From Equation (43), it seems that the contraction coefficient depends on the velocity of flow along the walls approaching the orifice. The pressure on this face will decrease as the velocity increases. If the value of the integral is known it is possible to compute the value of the coefficient.

This analysis has been made by assuming an ideal fluid not acted upon by gravity. According to experiment, with gravity acting, the fluid filaments are sensibly parallel a short distance outside the orifice and, hence, this factor has only a slight effect on the path of flow. As for the motion of the fluid within the basin, it is immaterial whether the flow is assumed to take place under the action of gravity or a constant pressure.

The presence of fluid friction in a real fluid does not materially change any of the conclusions drawn from Equation (43).

If a vertical weir in an open channel is assumed to be one-half the basin just considered, with the height, $\frac{b - a}{2}$, and the head, $\frac{a}{2}$, it is necessary to

measure the pressure on the face of the weir to determine the total effect of the velocity distribution on the value of μ. As such measurements have not previously been made in connection with measurements of the coefficient it is not possible to fix the total influence of the distribution.

The molecular forces may affect the discharge of a sharp-crested weir in one of three ways: First, either by adhesion, causing the nappe to cling to the crest, and giving the effect of its being rounded; second, by the surface tension acting on the surfaces of the sheet and giving a force in the same direction as the force of gravity, thereby causing an increase in the vertical acceleration of the discharge; and, third, by a combination of the other two.

If the rounding of the crest may be considered as constant for all heads, its influence will be greater at lower than at higher heads and the relation may be written,

$$\mu = f\left(\frac{\varepsilon}{h}\right) \qquad (44)$$

in which, ε is the radius of the crest caused by the adhesion. The effect of the surface tension may be taken into account by using the expression,

$$\mu = \chi\left(\frac{T}{\rho \, . \, g \, . \, h^2}\right) \qquad (45)$$

in which, T is the surface tension and $\rho\, g$ is the weight per unit volume. This function can be found by aid of dimensional analysis.

The influence of the molecular forces may be illustrated by Figs. 77 and 78, which show the same discharge over a sharp-crested weir, 700 mm. high, made of brass. The writer's experiments have shown that there will be no discharge over such a weir when $h = 0.0026$ m. When $h = 0.0028$ m., the discharge begins. If this is slowly increased, the sheet will cling to the face until the head is 0.059 to 0.062 m., when it will become free. By touching the sheet, or by introducing air below it, it can be freed when $h > 0.0105$.

Within this range it is possible to make the sheet cling by means of a board, etc. By slowly lowering the head, the sheet will remain free until $h = 0.010$. It will begin to cling at some point below this head and is always clinging at $h = 0.0088$ m. At heads greater than 0.062 m., the sheet will be free. These values are for a water temperature of 11° cent.

The writer's studies have shown that the coefficient of discharge, μ, is a function of the dimensionless quantities, $\frac{k}{h}$, δ, $\frac{\varepsilon}{h}$, and $\frac{T}{\rho\, g\, h^2}$, and that it will have the same value when each of the ratios is the same simultaneously.

With respect to the conditions expressed in Equation (38) the writer has studied the function and the value of δ by plotting known experimental values in a diagram, with μ as the ordinates and $\left[\mu \frac{h}{h + w}\right]^2$ as the abscissas. For this purpose the tests by the Swiss Hydrographic Bureau at Amsteg,* Bazin's experiments Nos. 5, 7, 9, and 10, and the Schoder and Turner Series D to O, have been chosen.

* Communications du Service des Eaux, No. 18, "Contribution à l'étude des méthodes de jaugeage".

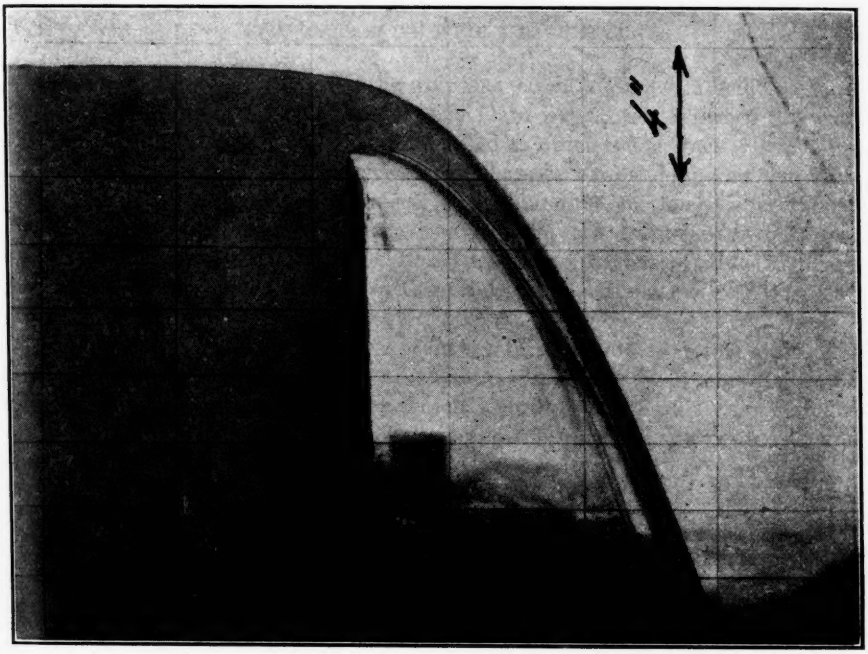

FIG. 77.—DISCHARGE OVER SHARP-CRESTED WEIR, $h > 0.0105$.

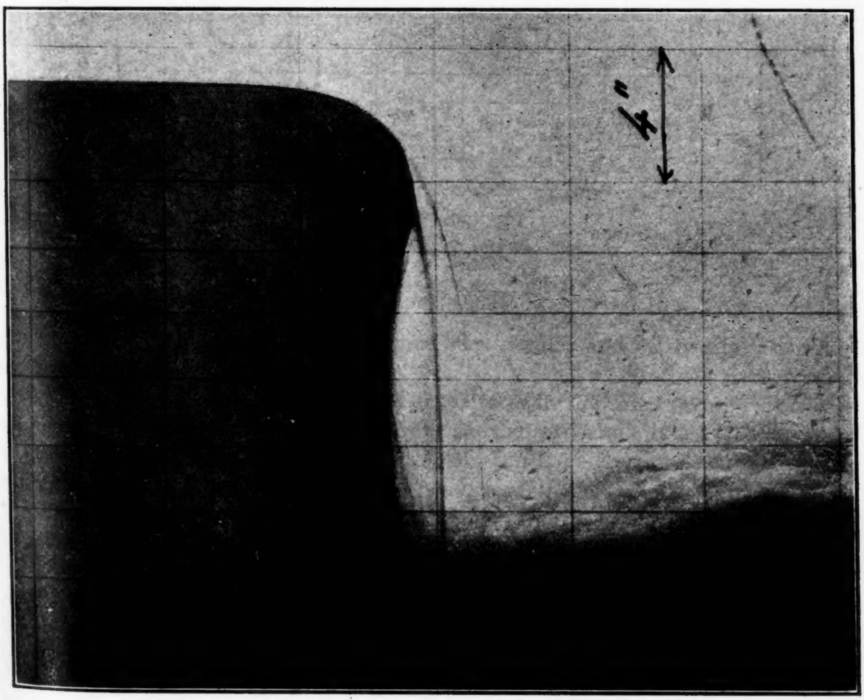

FIG. 78.—DISCHARGE OVER SHARP-CRESTED WEIR, $h < 0.0105$.

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In the experiments at Amsteg the weir was 0.8 m. high, 3.0 m. wide, and its distance from the draft-tube of the Amsteg plant was 23 m. Between the weir and the turbines thirteen racks were installed to quiet the surface.

The Bazin experiments were made in a channel 2 m. wide, using the same weir crest. The channel of approach was 44 and 95 m. long, respectively, measured below a calibrated, sharp-crested weir, the height of which was 1.135 m. The known fact that the Bazin tests give discharge values that are too high, does not affect the value of this study because the same weir was used as an indicator of the discharge.

Fig. 79 shows that the Bazin experiments give points that fall close to a straight line. The divergence of the points from the mean curve may be attributed to the experimental error, which according to the writer's experience affects the third decimal place. The average line may be represented by the formula,

$$\mu = 0.640 + 0.515 \left[\mu \cdot \frac{h}{h + w}\right]^2 \dots\dots\dots\dots\dots (46)$$

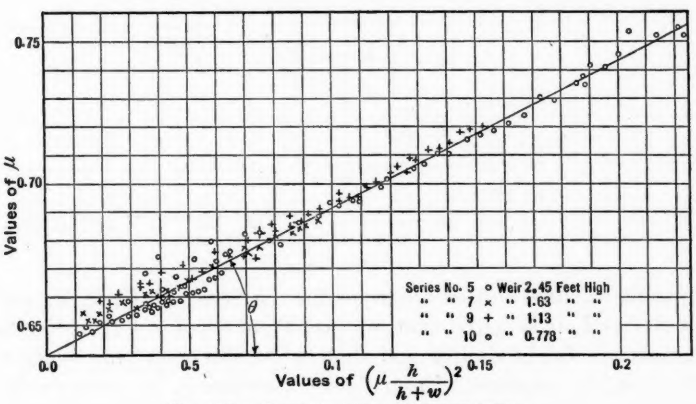

FIG. 79.—BAZIN'S WEIR EXPERIMENTS.

The computed value of δ is, therefore, $0.515 \times \frac{9}{4} = 1.16$. These results show that the distribution of velocity has been regular and fairly similar in all the experiments.

Before studying the Schoder and Turner tests in the same manner as those of Bazin, it is first necessary to examine the experiments on high weirs (Table 1*), particularly the Series D, J, K, L, M, and O, respectively, as to the relation between values of μ, the head, and the distribution of velocity. Fig. 80 shows that these series give the same value of μ within the limits of the probable experimental error.

The authors do not indicate the probable errors of the experiments in spite of the great assistance that such a computation gives for judging the reliability of experimental results. The formula for determining the error is obtained by taking the partial differential of Equation (21) which gives,

$$\frac{\delta Q}{Q} = \frac{\delta \mu}{\mu} + \frac{\delta L}{L} + \frac{3}{2} \frac{\delta h}{h} \dots\dots\dots\dots\dots (47)$$

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, pp. 1396–1397.

Since,

$$Q = \frac{\text{Volume}}{\text{Time}} = \frac{V}{T}$$

the error is given by,

$$\pm \frac{\delta \mu}{\mu} = \pm \frac{\delta V}{V} \mp \frac{\delta T}{T} \mp \frac{\delta L}{L} \mp \frac{3}{2} \frac{\delta h}{h} \dots\dots\dots\dots\dots (48)$$

in which each of the ratios on the right-hand side can be estimated from the precision of the instruments used or from the variation of individual readings for steady conditions.

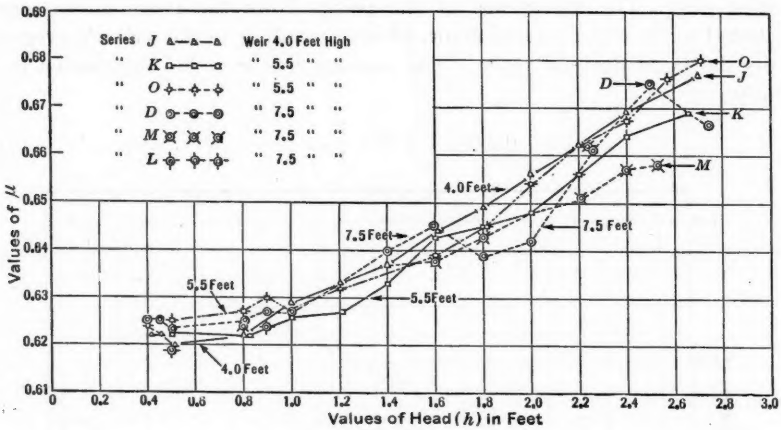

FIG. 80.

Comparing the distribution diagrams for the Series D, K, L, M, and O with those for Series J, it is found that, upward from a point 4 ft. below the crest, the diagrams are fairly similar and that below this point the depth has very little effect on the currents. From these experiments, it seems to the writer to be quite impossible to make a decision as to the applicability of any formula, since the results are so greatly influenced by the experimental arrangements used by the authors.

The Schoder and Turner Series E, F, and N are shown in Fig. 81. It will be noted that the points can be represented by a straight line, the slope of which is θ. In the lower range of the abscissas, the points diverge from this line by increasing amounts as the abscissas decrease. The three series of experiments give the formulas:

Series E and F,

$$\mu = 0.621 + 0.57 \left[\mu \frac{h}{h + w}\right]^2 ; \ \delta = 1.28 \dots\dots\dots\dots (49)$$

Series N,

$$\mu = 0.621 + 0.54 \left[\mu \frac{h}{h + w}\right]^2 ; \ \delta = 1.21 \dots\dots\dots\dots (50)$$

and the mean curve is,

$$\mu = 0.621 + 0.56 \left[\mu \cdot \frac{h}{h + w}\right]^2$$

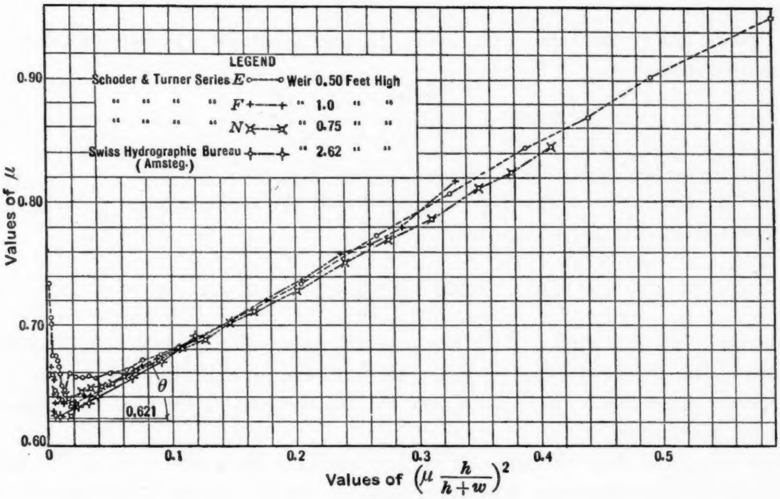
LEGEND
Schoder & Turner Series E Weir 0.50 Feet High
" " " " F " 1.0 " "
" " " " N " 0.75 " "
Swiss Hydrographic Bureau " 2.62 " "
(Amsteg.)
Values of μ
0.90
0.80
0.70
0.60
θ
0.621
0 0.1 0.2 0.3 0.4 0.5
Values of $\left(\mu \frac{h}{h+w}\right)^2$

FIG. 81.

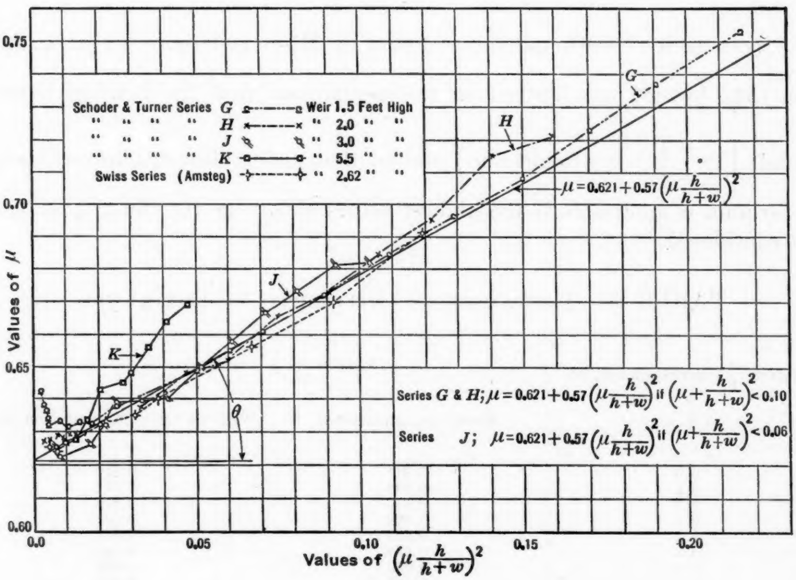
Schoder & Turner Series G Weir 1.5 Feet High
" " " " H " 2.0 " "
" " " " J " 3.0 " "
" " " " K " 5.5 " "
Swiss Series (Amsteg) " 2.62 " "
G
H
J
K
θ
$\mu = 0.621 + 0.57\left(\mu \frac{h}{h+w}\right)^2$
Series G & H; $\mu = 0.621 + 0.57\left(\mu \frac{h}{h+w}\right)^2$ if $\left(\mu + \frac{h}{h+w}\right)^2 < 0.10$
Series J; $\mu = 0.621 + 0.57\left(\mu \frac{h}{h+w}\right)^2$ if $\left(\mu + \frac{h}{h+w}\right)^2 < 0.06$
Values of μ
0.75
0.70
0.65
0.60
0.0 0.05 0.10 0.15 0.20
Values of $\left(\mu \frac{h}{h+w}\right)^2$

FIG. 82.

The greater the angle, θ, for any series, the larger will be the value of δ. This is illustrated in Fig. 82, representing the Schoder and Turner Series G, H, I, and K, and the experiments of the Swiss Hydrographic Bureau at Amsteg.

The Schoder and Turner Series G, H, and I can be represented fairly well by the formula,

$$\mu = 0.621 + 0.61 \left[\mu \frac{h}{h + w}\right]^2; \ \delta = 1.37 \ldots\ldots\ldots\ldots (51)$$

and the Swiss series can be as represented by Equation (50).

In this discussion it has been assumed that the experimental values can be represented by the formula,

$$\mu = 0.621 + \theta \left[\mu \frac{h}{h + w}\right]^2 \ldots\ldots\ldots\ldots\ldots\ldots (52)$$

Equation (52) will give $\mu = 1$ for $w = 0$, only when $\theta = 0.379$; and it is, therefore, limited in its application. As there are no experiments on low weirs under very high heads, it is necessary to find the proper form of the equation in another manner.

For the ideal flow through an orifice, such as discussed herein, Professor Mises, of Berlin, Germany, has computed the values of the coefficient of contraction by means of conformal transformations. His values (see Table 66) can be closely represented by the empirical formula,

$$\phi = 0.611 + 0.33 \left[\phi \frac{a}{b}\right]^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (53)$$

The agreement between the values found by Mises and those given by Equation (53) (which was derived on the assumption that the relation between ϕ and $\left[\phi \frac{a}{b}\right]^2$ is a straight line) shows, that under ideal conditions this type of formula is applicable if the highest values of ϕ near the limit, $\phi = 1$, are not considered.

TABLE 66.—COMPARISON OF COEFFICIENTS OF CONTRACTION.

Ratios, $\frac{a}{b}$, corresponding to $\frac{h}{h + w}$.	Values of ϕ According to: Mises' computations.	Writer's formula, Equation (52).
0.0	0.611	0.611
0.1	0.612	0.612
0.2	0.616	0.616
0.3	0.622	0.622
0.4	0.633	0.633
0.5	0.644	0.645
0.6	0.662	0.663
0.7	0.687	0.687
0.8	0.722	0.721
0.9	0.781	0.774
1.0	1.000	0.941

On account of this agreement and the foregoing results the writer believes that the type of formula, Equation (52), for sharp-crested weirs, fits its purpose well.

The influence of the molecular forces is expressed by the relation,

$$\mu = \phi \left\{ \frac{\varepsilon}{h}, \frac{T}{\rho\, g\, .\, h^2} \right\} \ldots\ldots\ldots\ldots\ldots\ldots (54)$$

The form of the function is not known. As the μ diverges from the straight line representing the formula for higher heads, the effect of the molecular forces may be determined by subtracting the μ given by Equation (52) from the observed value.

$$\mu_{\text{Remainder}} = \mu - \theta \left[\mu \frac{h}{h + w} \right]^2 - 0.621$$

Plotting these values against the head gives, in foot-pounds,

$$\mu_{\text{Remainder}} = \frac{0.0007}{h^{\frac{3}{2}}}$$

which represents the results of Schoder and Turner Series E, F, G, H, J, and N (see Fig. 83).

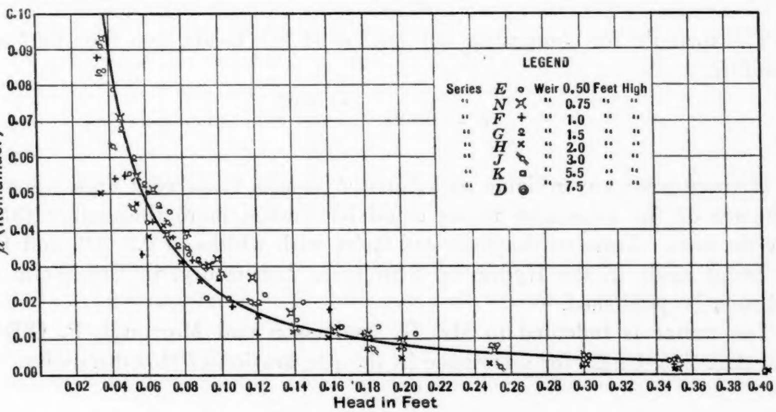

FIG. 83.

Since μ is a number, the numerator, 0.0007, has the dimensions, $l^{\frac{3}{2}}$, including the surface tension, T, which, in common with other factors, depends on the temperature.

On the basis of the Swiss, Bazin, and the Schoder and Turner tests, the formula for computing the dicharge coefficient for a sharp-crested weir is,

$$\mu = 0.621 + \theta \left[\mu \frac{h}{h + w} \right]^2 + \frac{0.0007}{h^{\frac{3}{2}}} \ldots\ldots\ldots\ldots\ldots (55)$$

or,

$$\mu = \frac{1}{2\,\theta} \left(\frac{h + w}{h} \right)^2 \left\{ 1 - \sqrt{1 - 4\,\theta \left(0.621 + \frac{0.0007}{h^{\frac{3}{2}}} \right) \left(\frac{h}{h + w} \right)^2} \right\} \ldots (56)$$

in which, θ has the following values,

Schoder and Turner Series E and F	0.57
" " " " N	0.54
" " " " G, H, and J	0.61
Swiss Series Amsteg	0.54
(Bazin's Series Nos. 5, 7, 9, and 10	0.515)

The formula is rather complicated, but for practical use it can be represented graphically.

When the head on the weir is low, the velocity of approach has very little influence on the μ-value. Assuming, then, 0.001 as an upper limit for the error, ignoring the velocity head term and making $\theta = 0.55$ and $\mu = 0.62$, the resulting relation between the head and the height of the weir, at which the velocity of approach can be neglected, is:

$$\theta \left[\mu \frac{h}{h + w}\right]^2 < 0.001 \quad \text{.....................} (57)$$

Hence,

$$\frac{h}{w} < 0.07$$

The formula for computing the μ-value at low heads, less than 0.07 w, is, therefore,

$$\mu = 0.621 + \frac{0.0007}{h^{\frac{3}{2}}} \quad \text{.....................} (58)$$

If there were any reliable experimental results from very high weirs, the influence of the molecular forces could be studied more thoroughly than is now the case. Tests on sharp-crested weirs with widths of 0.5, 2.0, and 10 ft. are being made in the Hydraulic Structures Laboratory, in Stockholm, and will soon be published.

The writer is indebted to Mr. H. Andersson and Morrough P. O'Brien, Jun. Am. Soc. C. E., for assistance in the preparation of this discussion.

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PAPERS AND DISCUSSIONS

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NEW THEORY FOR THE CENTRIFUGAL PUMP

Discussion*

By A. B. Cox, Esq.

A. B. Cox,† Esq. (by letter).‡—Any one who has given any study to the history of science and engineering cannot but be impressed with the way in which new theories and practices have been—and are—constantly overthrowing old theories and practices, only to be themselves overthrown by still other theories and practices later. A thousand years or so from now, practically all the present theories and practices will certainly have been superseded or radically changed; but why wait a thousand years? Why not encourage—rather than discourage—such men as Professor Sherzer, who are trying, now, to discover the errors in old theories and to devise others, better and more workable?

In Fig. 35 is shown a series of head-capacity curves of a steam-condenser circulating water pump with low head and high capacity, and in Fig. 36 a series for an oil-cooler circulating water pump with high head and low capacity. A notable feature of the curves of Fig. 35 is the **V**, or notch, near the middle of each curve. This droop in the curve was not noticed during the test until the last curve was being taken—at 720 rev. per min.—whereupon the points at this speed were carefully checked and found to be correct. It was then discovered that similar notched curves could be drawn through the points of all the other curve points taken at the lower speeds. The droop in the head-capacity curve appears to be characteristic of centrifugal pumps and fans.§ Probably the chief reason that this peculiarity has not been noticed before is that most pumps and fans have only a slight droop in the head-capacity curve which is scarcely noticeable, and the points of the curve are usually taken on test only at the shut-off point and near the rating point, the section in which the **V** is most likely to occur nearly always being omitted.

* Discussion of the paper by A. F. Sherzer, Assoc. M. Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Designer, R. D. Nuttall Co., Pittsburgh, Pa.

‡ Received by the Secretary, April 27, 1928.

§ "Performance of Centrifugal Fans for Electrical Machinery," by C. J. Fechheimer, *Transactions*, Am. Soc. Mech. Engrs., Vol. 46 (1924), p. 287.

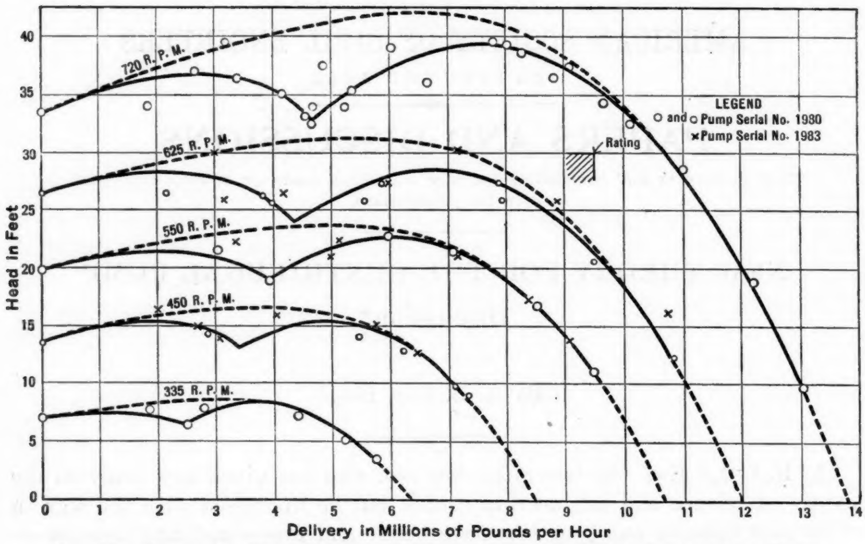

FIG. 35.—SERIES OF HEAD-CAPACITY CURVES, MAIN CONDENSER CIRCULATING PUMP.

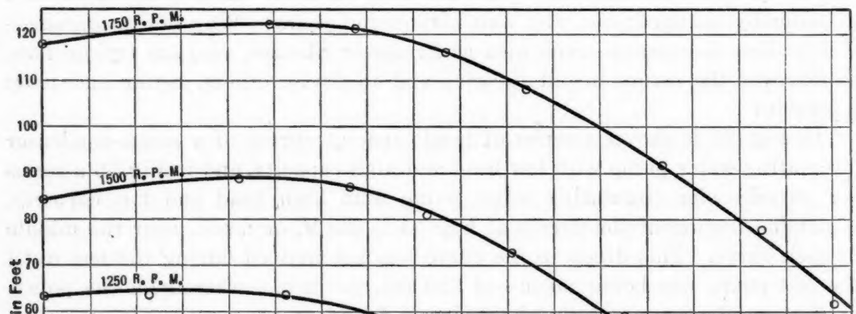

FIG. 36.—SERIES OF HEAD-CAPACITY CURVES, OIL COOLER CIRCULATING WATER PUMP SERIAL No. 1984.

The diameters of the pump runner of Fig. 35 were 19 in. outside and $14\frac{9}{16}$ in. inside, and that of Fig. 36, $11\frac{5}{8}$ in. and $5\frac{1}{2}$ in., respectively. The values of the head at shut-off, as calculated by the formula, $\frac{V^2}{g}$, for the two pumps, are 110.6 ft. and 245 ft. at speeds of 720 and 1 750 rev. per min., respectively, instead of 33.5 ft. and 117.5 ft. as actually obtained on test. The values calculated by the formula, $\frac{V^2}{2\,g}$, are 55.3 ft. and 122.5 ft. respectively.

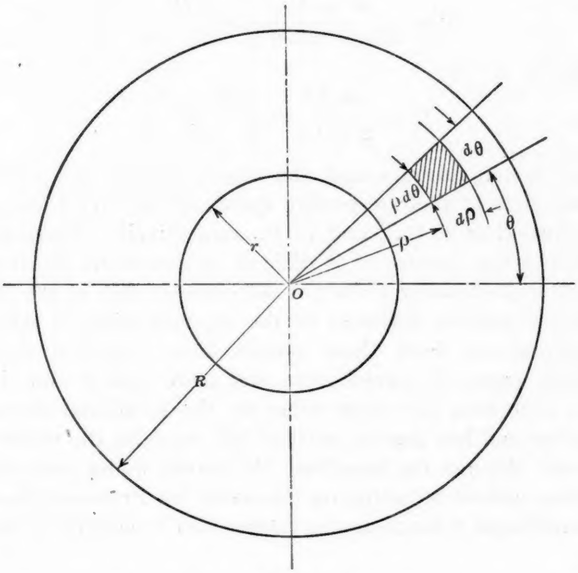

FIG. 37.

Another way of considering the action of a centrifugal pump at shut-off is to assume the impeller to be a rotating hollow disk or cylinder of water. In Fig. 37, let R be the outer radius, in feet, and r the inner radius of a cylinder of water, l ft. long, rotating at a speed of N rev. per min. about its axis at Point O. The volume of the differential element shown in the diagram is, therefore, $d\,v = l\,\rho\,d\,\theta\,d\,\rho$, and the weight, in pounds, of the element is $d\,W = w\,l\,\rho\,d\,\theta\,d\,\rho$, in which, w is the weight, in pounds, of 1 cu. ft. of fluid. The formula for centrifugal force acting on a rotating body of weight, W, rotating at a speed of N rev. per min., at a distance of R_c ft. from the axis of rotation is $F = \frac{W\,V^2}{g\,R_c}$ lb. For the element considered in Fig. 37 the velocity, V, is, $V = \frac{2\,\pi\,\rho\,N}{60}$ ft. per sec., $R_c = \rho$, and $W = d\,W$. Substituting these values in the centrifugal force formula:

$$F = \int_0^{2\,\pi} \int_r^R \frac{4\,\pi^2\,\rho^2\,N^2}{3\,600} \times \frac{w\,l\,\rho\,d\,\theta\,d\,\rho}{g\,\rho}$$

Simplifying, integrating, and substituting the proper limits:

$$F = \frac{2\,\pi^3\,w\,l\,N^2}{2\,700\,g}\,(R^3 - r^3)\text{ lb.}$$

the total pressure acting at the circumference of the cylinder. The area of the cylindrical surface, in square inches, is $A = 144 \times 2\,\pi\,R\,l$. The pressure at the periphery, in pounds per square inch, is $P = \frac{F}{A}$, and substituting the values of F and A,

$$P = \frac{\pi^2\,w\,N^2}{144 \times 2\,700\,g} \times \left[\frac{R^3 - r^3}{R}\right]$$

and the head, is,

$$H = \frac{\pi^2\,N^2}{2\,700\,g} \times \left[\frac{R^3 - r^3}{R}\right]$$

Using this latter formula and the exact outside and inside diameters of the two impellers at their respective speeds of 720 and 1 750 rev. per min., the head at shut-off is 29.4 ft. and 72 ft., respectively. These results do not check the tests either, because it is difficult to determine the inside diameter of the rotating disk of water (it is not the same as that of the impeller), and to determine the outside diameter of the rotating disk, if the pump has a whirlpool chamber—as both these pumps have. As the discharge valve of a centrifugal pump is opened more and more widely and the volume of the discharge increases, the water takes up the rotational movement of the impeller to a less and less degree, until at full capacity the water flows almost radially outward through the impeller. Of course, under such circumstances, centrifugal force cannot be acting on the water (as Professor Sherzer's results show), and centrifugal force formulas cannot well be applied to the calculation of the head.

The writer has also endeavored to devise a theory for the centrifugal pump —a theory which should account for the **V**-notches in the head-capacity curves of Fig. 35, and which should enable these curves to be predicted from the pump measurements, as Professor Sherzer has done in Figs. 9,* 10,* and 11.† The writer's endeavor has not yet been successful, but it has resulted in "A New Theory of Fluid Flow",‡ which may be of interest to pump designers and to engineers studying the flow of water, oil, or air, and to those interested in the thickness of lubricating films and in heat transmission through fluids.

There do not seem to be sufficient detail of design and test data in Professor Sherzer's paper to enable one to check from this information alone whether or not some of the chief points of the theory are correct. This is a minor omission, for every pump designer should have sufficient detailed pump data of his own available for checking the theory independently of Professor Sherzer's data—which, after all, is a preferable method of checking; but regardless of whether the theory is, or is not, correct, the data that are given would seem to be sufficient to show that here is a new method of pump

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1793.

† *Loc. cit.*, p. 1794.

‡ *Journal*, Franklin Inst., December, 1924, and December, 1926.

design which is capable of being applied in some cases to obtain excellent results, such as are shown in Figs. 9, 10, 11, 15,* and 16.† Also, the author's conclusions in regard to guide-vanes are supported by the recently published tests of Mr. H. F. Schmidt.‡

The centrifugal pump is a machine of complicated action. It has pressure effects and velocity effects; it has centrifugal and other inertia effects; it has frictional and eddy effects; and if poorly designed, it may have sudden droops in the head-capacity curve, or vacuum pockets formed in the impeller, similar to cavitation effects in ship propellers, etc. In an effort to design a simpler pump and to apply what has been learned from analysis of pumps having head-capacity curves as shown in Figs. 35 and 36, the writer has been so fortunate as to have developed a radically new but simple type of axial flow pump. The first tests indicate that the weaknesses typical of the usual design of this type of pump have been avoided and the good points of the centrifugal pump—particularly for high-head work—have been retained and improved upon. It is hoped that testing will be completed some time during 1928 so that the details of the design can be made public and thus add a little more to the rapidly growing knowledge of pump design.

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* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1797.

† *Loc. cit.*, p. 1798.

‡ "Some Screw Propeller Experiments with Particular Reference to Pumps and Blowers," *Journal*, Am. Soc. of Naval Engrs., February, 1928.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

BAFFLE-PIER EXPERIMENTS ON MODELS OF PIT RIVER DAMS

Discussion*

By Messrs. Paul Bauman and Morrough P. O'Brien.

Paul Bauman,† M. Am. Soc. C. E. (by letter).‡—The prevention of scouring of river beds and banks, caused by over-pour water from a weir, was a problem to engineers for many years and, although innumerable respective devices were created, none of them really deserved to be called a solution. Misconceptions of the subject helped greatly to delay a satisfactory solution, the principal one being the failure to differentiate between the effect of impact of falling water and the effect of a change of the distribution of water-thread velocities as related to their distribution for the normal flow of the river.

Judging from the description of the results of experiments on models of Pit River Diversion Dams Nos. 3 and 4, respectively, it is safe to state that the authors have found a solution, thereby contributing a valuable addition to the knowledge of practical engineers as well as a scientific "nut to be cracked" by those who are interested in the "why" of hydraulic puzzles. Whether or not the arrangement of baffle-piers, as finally chosen by the authors, will prove as successful for an actual flood of 70 000 cu. ft. per sec. as it did for a corresponding discharge over the model dams, is a question which only experience can answer definitely.

The writer, however, believes that, as far as the stilling action is concerned, no appreciable difference between the model and the actual conditions will show unless pulsations should develop due to disturbances in the unprotected river channel below the dam. It must be realized that the natural process of bed-forming in a river is handicapped by any artificial obstruction, such as a weir or a dam, inasmuch as its main factor—the transportation of débris—is more or less interrupted.

* Discussion of the paper by I. C. Steele and R. A. Monroe, Members, Am. Soc. C. E. continued from May, 1928, *Proceedings*.

† Designing Engr., Quinton, Code and Hill, Los Angeles, Calif.

‡ Received by the Secretary, March 30, 1928.

Before any hydraulic computation on the weir itself is done, it is, therefore, quite important to study the behavior of the river in its natural bed some distance below. A river flowing on alluvium forms its slope to suit itself. Governing in the formation of the bed is the flow for which the volume of transported débris is a relative maximum. The respective depth is often called the "bed-forming depth" and the respective flow likewise "the bed-forming flow." The hydraulic radius, r, and the slope, s, for this flow are the main characteristics of a river. At low stage, a river has the tendency to raise its bed, whereas at high stage, and especially at flood, it has a lowering or a scouring effect. It must be understood, therefore, that scouring takes place continuously (except for very low stages) regardless of whether or not there is a weir.

Let m = mass of the water;
v = velocity of water threads acting on the surface of a resisting body;
A = surface area of the body;
w = unit weight of water (62.5 lb. per cu. ft.);
μ = coefficient of friction;
V = volume of the water;
γ_0 = unit weight of the body, corrected for uplift; and,
K = a coefficient depending on the shape of the exposed surface, A.

Then, the kinetic force or impact of flowing water acting on a resisting body, such as a particle of sand, a piece of gravel, a pebble, or a boulder, is equal to,

$$F = 1\,P = K\,m\,v \quad \text{.......................} \quad (1)$$

and, since $m = \dfrac{w\,A\,v}{g}$,

$$P = \frac{K\,w\,A\,v^2}{g} \quad \text{.......................} \quad (2)$$

and the resistance, R, of the body is equal to,

$$R = \mu\,V\,\gamma_0 \quad \text{.......................} \quad (3)$$

When $P = R$, motion of the body is impending. Therefore, combining Equations (2) and (3) and solving for v:

$$v = \sqrt{\frac{V\,\gamma_0\,g\,\mu}{K\,A\,w}} \quad \text{.......................} \quad (4)$$

which is the velocity required to move the body; that is, the water-thread velocity at the bottom of the river. Observations will show, however, that much greater volumes of débris are moved at times than are given by Equation (4). This is due to the fact that, beside the scouring force, P, a force, D, is active as shown in Fig. 52.

Let f = area of cross-section of a water column;
$s \doteq \sin\alpha = \dfrac{h'}{l}$ = slope of the water surface;
d = depth of the stream; and
μ' = average coefficient of friction.

Then, according to the law of energy,

$$l \times D = w\, d\, f\, h' \quad \text{.........................(5)}$$

If f is made equal to 1 sq. ft.,

$$D = 62.5\, d\, s \quad \text{..........................(6)}$$

which is called the dragging force. The resistance, R', of 1 sq. ft. of river bed (see Fig. 53) is,

$$R' = \gamma_0\, t\, \mu' \quad \text{..........................(7)}$$

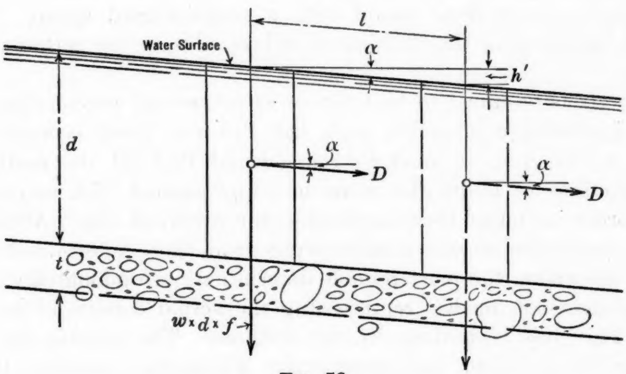

FIG. 52.

and when D is equal to R', Equations (6) and (7) may be combined to determine the depth, t, of the river bed that is in motion, or the limiting size, t, of the pebbles or boulders moved. This is equal to,

$$t = \frac{62.5\, d\, s}{\gamma_0 \times \mu'} \quad \text{..........................(8)}$$

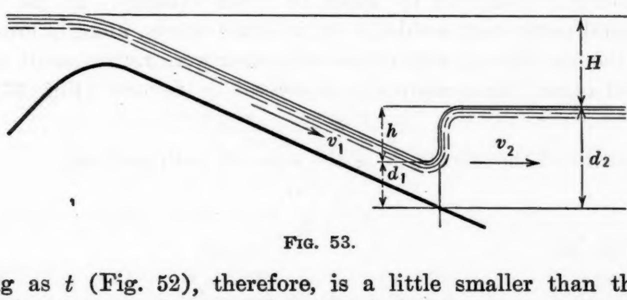

FIG. 53.

As long as t (Fig. 52), therefore, is a little smaller than the smallest diameter of aggregates forming the river bed, no motion will occur. This state of inertia corresponds to a flow during which neither scouring nor settling of débris occurs. It applies particularly to rivers with stony beds, such as the Pit River, because sandy beds are seldom at rest. The abrupt change in pressure between the water in the upper layers of a sandy river bed and the relatively fast-moving water threads immediately above has a lifting effect on the top particles which, once loosened, are readily carried away. The vertical velocity corresponding to this change in pressure has been observed to be from one-tenth to one-thirtieth of the horizontal velocity.

It is quite important to design the weir with due respect to the natural conditions of the river. The main source of scouring of over-pour water only too often lies in a poor adjustment of the weir to the natural bed of the river and above all of the apron. Weir construction is done as often as possible at low water and the temptation to put the apron on top of the then existing bed with a minimum of excavation is considerable. The bed-forming flow alone sometimes will lower the bed enough to cause the water to jump off the high apron, producing impact and eddies and probably doing more damage than a great flood would with a well-designed apron. The aforementioned handicap to the shifting of débris adds to the seriousness of this occurrence.

If the weir is assumed to be built so as to prevent percolation, it is governed by hydrodynamic, static, and, last but not least, economic features only. As to the first, it must be remembered that all the mathematically derived equations of hydraulics must be supplemented with empirical coefficients in order to make them applicable for practical use. Although these coefficients may often be chosen successfully from those determined for similar conditions elsewhere, the most reliable determination, undoubtedly, will result from experiments on models representing the actual conditions as closely as possible. This has been done by the authors. The greater the velocities involved in the problem, the greater the discrepancy between the strictly theoretical and the actually observed conditions and, therefore, the greater will be the importance of experiments because little is known about the "rushing or shooting" flow; that is, the flow at an average velocity, v, greater than wave velocity, v_w, inasmuch as ordinary hydraulic formulas strictly apply to "quiet" flow; that is, for $v < v_w$ only.

The discharge from a weir usually represents the case where water of higher velocity is absorbed by water of lower velocity. If the changes of velocity would occur very suddenly or, in other words, along a cross-sectional plane and the absolute velocity of the tail-water were rather small, a hydraulic jump would occur, mathematically expressed as follows (Fig. 53):

$$v_1\, d_1\, b = v_2\, d_2\, b \qquad (9)$$

If the width of the stream, b, is the same at both sections,

$$v_2 = \frac{v_1\, d_1}{d_2} \qquad (10)$$

From Fig. 53,

$$h = d_2 - d_1 = \frac{v_1^2 - v_2^2}{2\, g} \qquad (11)$$

Substituting the value of v_2 in Equation (10),

$$d_2 - d_1 = \frac{v_1^2\, (d_2^2 - d_1^2)}{2\, g\, d_2^2} \qquad (12)$$

By algebra,

$$d_2^2 - d_1^2 = (d_2 + d_1)\, (d_2 - d_1) \qquad (13)$$

Substituting Equation (13) into Equation (12) and solving for:

$$d_2 = \frac{v_1^2}{4\, g} \pm \sqrt{\frac{v_1^2}{2\, g}\left(\frac{v_1^2}{8\, g} + d_1\right)} \qquad (14)$$

which is a well-known formula. Note that, since d_2 cannot become negative, the sign before the radical is always plus. If the discharge is altered so as to diminish the height, h, of the jump until it finally becomes zero, then $d_2 = d_1$. Substituting d_1 for d_2 in Equation (14) and solving for v_1:

$$v_1 = \sqrt{g\,d_1} = v_w \qquad (15)$$

or,

$$d_1 = \frac{v_1^{\,2}}{g} = 2\,h_1$$

This means that in order to produce a jump, the velocity head, h_1, must be at least as great as one-half the depth, or the velocity, v_1, must be at least equal to the wave velocity, v_w. Another condition for the elimination of a jump follows from,

$$h = \frac{v_1^{\,2} - v_2^{\,2}}{2\,g} \qquad (16)$$

in which, h becomes zero for $v_1 = v_2$ or, in other words, no jump occurs if the tail-water velocity is equal to or greater than the velocity of the over-pour water in the plane of contact. Neither the latter condition nor the one for a hydraulic jump, however, is ordinarily present where a flood discharge over a weir in a (natural) river is concerned, but one or more standing waves may form as an "outburst" of kinetic energy accumulated in the falling water, due to a more or less sudden change in slope. While, in principle, the standing wave is nothing but a modified jump, its height is not determinable by means of Equation (14) for weirs with a relatively high drop, H (Fig. 54), and only approximately, for those with a relatively low drop. The reason is that only part of the kinetic energy is transformed into potential energy while the remainder is transformed into thermal energy due to the formation of a superficial eddy. Inasmuch as this energy is non-injurious to either the protected or the unprotected river bed, it is evident that the tendency must be to decrease the undesirable standing wave and to encourage the formation of a large superficial eddy. In principle, this may be accomplished by means of a smooth transition between the down-stream slope of the weir and the apron (Fig. 54) and by the proper width, length, and elevation of the latter or, in other words, by providing a sufficient depth, d_2. The efficiency of this eddy depends on the ratio of $\frac{H}{d_2}$ (Fig. 54), which is best determined on models.

The absorption of kinetic energy takes place to such a degree that the water emerging from beneath the superficial eddy always flows with an average velocity that does not exceed the wave velocity. The volume, in cubic feet, of a superficial eddy required to perform this transformation of energy has been determined by Professor Theodor Rehbock* as being between,

$$V_E = 3.60\,Q\sqrt{\frac{H}{g}} \qquad (17)$$

* "Betrachtungen über Abfluss, Stau-und Walzenbildung bei fliessenden Gewässern", Berlin, 1917, Jul. Springer.

and,

$$V_E = 7.20\ Q \sqrt{\frac{H}{g}} \text{......................} (18)$$

in which, Q equals discharge, in cubic feet per second, and H equals drop (see Fig. 54).

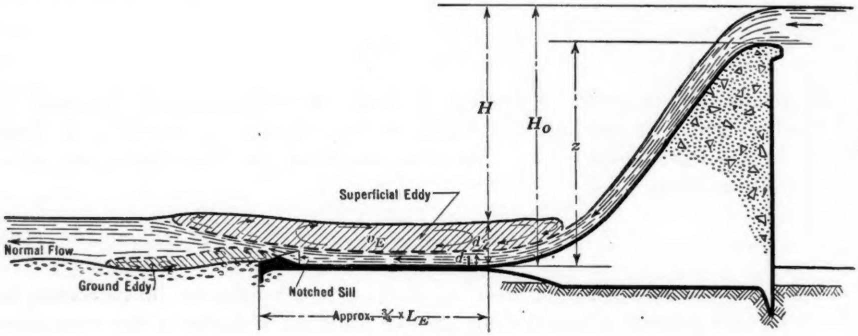

FIG. 54.

The two limits of Equation (17) indicate that an efficiency is involved which, considering the fact that the superficial eddy is transforming energy, is quite plausible.

If K = a constant, corresponding to 3.60 in Equation (17);
Q' = discharge per unit width of channel;
η = efficiency in percentage; and
H_0 and d_2 are as shown in Fig. 54;

Equation (17) may be written,

$$V_E' = \frac{K}{\eta}\ Q' \sqrt{\frac{H_0 - d_2}{g}} \text{......................} (19)$$

The efficiency is then given by:

$$\eta = \frac{K\ Q'}{V_E'} \sqrt{\frac{H_0 - d_2}{g}} \text{......................} (20)$$

It is evident that the efficiency, η, can be determined on models by computing the numerator, assuming that Q' and H_0 are constant and that d_2 is variable, and by measuring the respective cross-sectional area, V_E'. Plotting the resulting values of η as ordinates and the corresponding values of d_2 as abscissas will give points on a curve such that the maximum indicates the desirable depth, d_2, which should be proportionally applied to the weir as closely as conditions permit.

Let $K' = \dfrac{K}{\eta}$, which is constant for a small change of H and v_2;

c_p = specific heat of water;
ΔT = increase of temperature within the superficial eddy;
L_E = effective length of the superficial eddy;

Q'' = discharge per linear foot of weir $= \dfrac{B}{b}\, Q'$;
b = length of weir crest;
B = width of apron;
z = height of weir crest above apron;
c = coefficient of weir discharge;
a = a constant $= 777.64\ K'\, 2g$; and
1 B. t. u. = 777.64 ft-lb.

Then,

$$K'\ Q' \sqrt{\frac{H}{g}}\ w\ c_p\ \Delta\ T = \frac{1}{777.64}\left(w\ Q'\ H - \frac{m\ v_2^{\,2}}{2}\right) \qquad (21)$$

Since $m = \dfrac{w\ Q'}{g}$ and $c_p = 1$, Equation (21) may be written (neglecting the energy necessary to overcome friction on the way, L_E),

$$K'\ \Delta\ T \sqrt{\frac{H}{g}} = \frac{1}{777.64}\left(H - \frac{v_2^{\,2}}{2\ g}\right) \qquad (22)$$

and,

$$\frac{777.64\ K'\ 2\ g}{\sqrt{g}}\ \Delta\ T = 2\ g\ \sqrt{H} - \frac{v_2^{\,2}}{H} \qquad (23)$$

Substituting the value, a:

$$\Delta\ T = \frac{1}{a}\left(2\ g\ \sqrt{H} - \frac{v_2^{\,2}}{H}\right) \qquad (24)$$

Considering that $v_2 = \dfrac{Q'}{d_2}$, $Q' = \dfrac{b}{B}\ Q''$, and Equation (21) can be solved into the form:

$$\Delta\ T = \frac{2\ g}{a}\left(\sqrt{H} - \frac{b^2\ c^2\ (H + d_2 - z)^3}{B^2\ d_2^{\,2}\ \sqrt{H}}\right) \qquad (25)$$

The total derivation of ΔT by the parameters, H and d_2, is given by:

$$\delta\ (\Delta\ T) = \frac{\delta\ (\Delta\ T)}{\delta\ H}\ d\ H + \frac{\delta\ (\Delta\ T)}{\delta\ d_2}\ d\ (d_2) \qquad (26)$$

and the superficial eddy is most effective if ΔT becomes a maximum, which means that $\dfrac{\delta\ (\Delta\ T)}{\delta\ H} = 0$ and $\dfrac{\delta\ (\Delta\ T)}{\delta\ (d_2)} = 0$, simultaneously, and,

$$\frac{\delta\ (\Delta\ T)}{\delta\ (H)} = 0 = \frac{1}{\sqrt{H}}$$

$$-\frac{d_2^{\,2}\ \sqrt{H}\ B^2\ 3\ c^2\ (H + d_2 - z)^2\ 1\ b^2 - c^2\ (H + d_2 - z)^3\ b^2\ \dfrac{d_2^{\,2}}{\sqrt{H}}\ B^2}{d_2^{\,4}\ H\ B^4} \qquad (27)$$

$$0 = d_2^{\,2}\ B^2 - b^2\ 3\ c^2\ (H + d_2 - z)^2 + b^2\ c^2\ (H + d_2 - z)^3\ \frac{1}{H} \qquad (28)$$

$$\frac{\delta\ (\Delta\ T)}{\delta\ (d_2)} = 0$$

$$= -\frac{d_2^{\,2}\ \sqrt{H}\ B^2\ 3\ c^2 (H + d_2 - z)^2\ 1\ b^2 - c^2\ (H + d_2 - z)^3\ b^2\ 2\ d_2\ \sqrt{H}\ B^2}{d_2^{\,4}\ H\ B^4} \qquad (29)$$

$$0 = -d_2\, 3\, c^2\, (H + d_2 - z)^2\, b^2 + 2\, c^2\, (H + d_2 - z)^3\, b^2 \quad \text{........} (30)$$

By adding Equations (28) and (30), substituting $\frac{(Q'')^2}{2\,g}$ for $c^2\,(H + d_2 - z)^3$, and simplifying the equation, the result is:

$$d_2^{\,3} = \frac{(Q'')^2\, b^2}{g\, B^2} \left(1 - \frac{d_2}{2\,H}\right) \quad \text{...................} (31)$$

When $H = \frac{d_2}{2}$, the quantity, $1 - \frac{d_2}{2\,H}$, becomes zero, which indicates at once that it is a limiting condition because $\frac{(Q'')_2\, b_2}{g\, B^2}$ then becomes $+\infty$, whereas, for $H = +\infty$, the quantity, $1 - \frac{d_2}{2\,H}$, becomes $+1$ and $d_2 = \sqrt[3]{\frac{(Q'')^2\, b^2}{g\, B^2}}$.

The result of Equation (31) is shown in Fig. 55. At Pit No. 3 Dam, $Q'' = \frac{70\,000}{267} = 262$ cu. ft. per sec.; $b = 267$ ft.; $B = 200$ ft., $z = 102$ ft.; and $H_0 = 120$ ft. Therefore, $H = 104.75$ ft. and d_2 (by Equation (31)) $= 15.25$ ft. At Pit No. 4 Dam, $Q'' = \frac{70\,000}{136} = 515$ cu. ft. per sec.; $b = 136$ ft., $B = 160$ ft., $z = 40$ ft. $\pm$; and $H_0 = 67$ ft. $\pm$. Under these conditions, $H = 50.00$ ft. and $d_2 = 17.00$ ft.

As to the actual conditions, it is estimated that for $Q = 70\,000$ cu. ft. per sec., $d_2 = 17.00$ ft., and $H = 103.00$ ft., at Pit No. 3 Dam, and $d_2 = 21.00$ ft., and $H = 46.00$ ft., at Pit No. 4 Dam, which shows enough similarity to the depth, d_2, and the drop, H (as given by Fig. 55), to warrant a very effective superficial eddy. In computing its volume, therefore, it seems permissible to use,

$$V_E = 3.60\; Q \sqrt{\frac{H}{g}}$$

For Pit No. 3 Dam,

$$V_E = 3.60 \times 70\,000 \sqrt{\frac{103}{32.20}} = 452\,000 \text{ cu. ft.}$$

or a cross-sectional area of,

$$A_E = \frac{452\,000}{200} = 2\,260 \text{ sq. ft.}$$

The mean depth, $d_E = d_2 - d_1$ (Fig. 54). As (Fig. 56) d_1 is, roughly, 4 ft., $d_E = 17 - 4 = 13$ ft., which gives an effective length,

$$L_E = \frac{A_E}{d_E} = \frac{2\,260}{13} = 175 \text{ ft., more or less.}$$

Similarly, for Pit No. 4 Dam,

$$V_E = 3.60 \times 70\,000 \sqrt{\frac{46}{32.20}} = 300\,000 \text{ cu. ft.}$$

$$A_E = \frac{300\,000}{160} = 1\,875 \text{ sq. ft.}$$

$$d_E = d_2 - d_1 = 21 - 8 = 13 \text{ ft.}$$

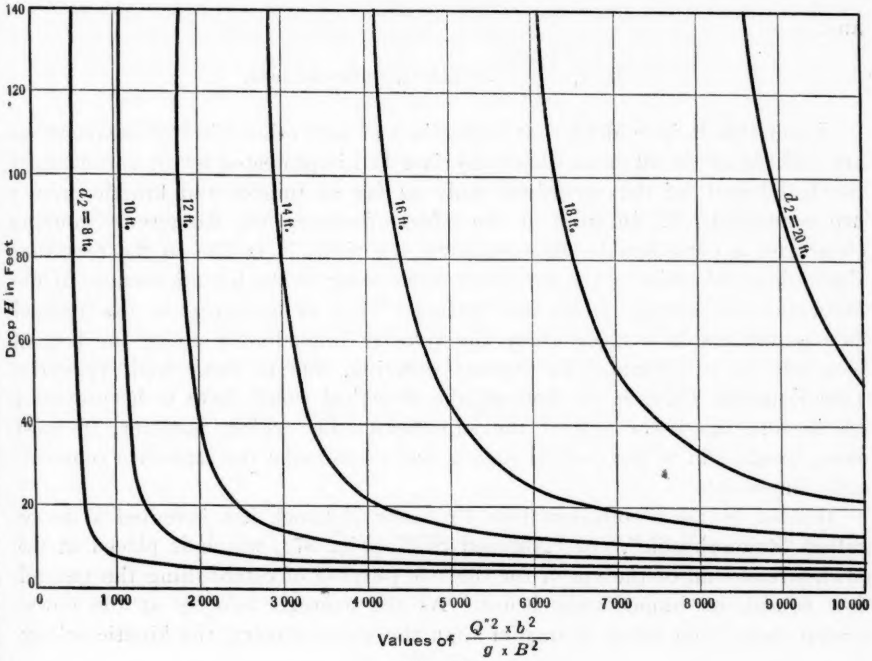
Drop H in Feet
140
120
100
80
60
40
20
0
0
1 000
2 000
3 000
4 000
5 000
6 000
7 000
8 000
9 000
10 000
$d_2 = 8$ ft.
10 ft.
12 ft.
14 ft.
16 ft.
18 ft.
$d_2 = 20$ ft.
Values of $\frac{Q''^2 \times b^2}{g \times B^2}$

FIG. 55.

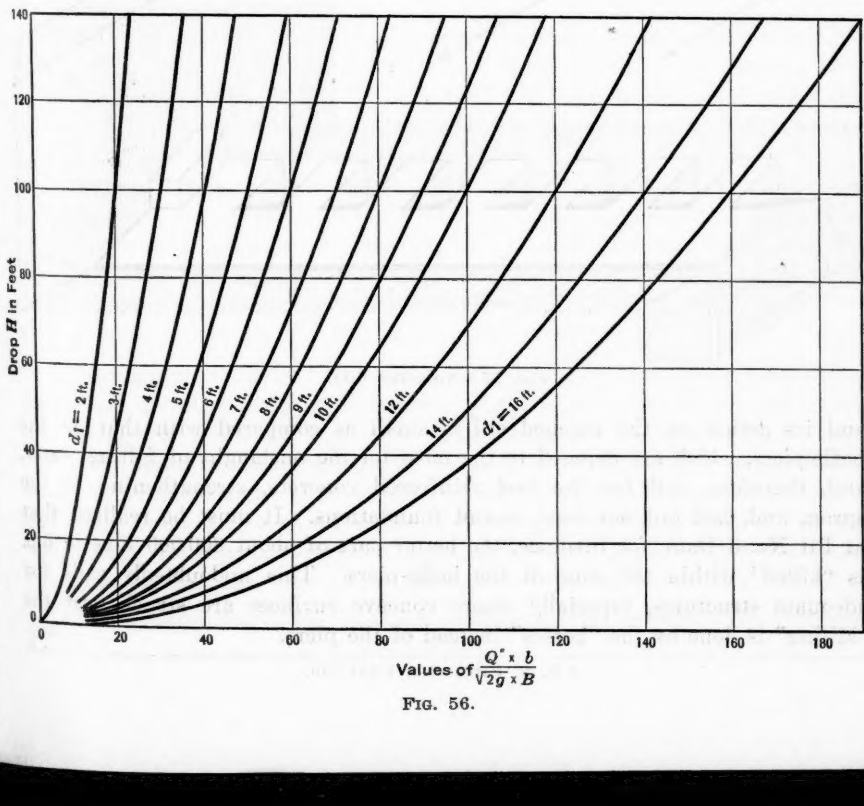
Drop H in Feet
140
120
100
80
60
40
20
0
0
20
40
60
80
100
120
140
160
180
$d_1 = 2$ ft.
3 ft.
4 ft.
5 ft.
6 ft.
7 ft.
8 ft.
9 ft.
10 ft.
12 ft.
14 ft.
$d_1 = 16$ ft.
Values of $\frac{Q'' \times b}{\sqrt{2g} \times B}$

FIG. 56.

and,

$$L_E = \frac{1\,875}{13} = 145 \text{ ft., more or less.}$$

From this it is evident that baffle-piers or any other kind of obstructions are superfluous in all cases where the river bed is protected by an apron under the full length of the superficial eddy as far as impact and kinetic energy are concerned. If, in spite of the effect of absorption, dangerous scouring occurs for a considerable distance down the river, it is due to the fact that the highest velocities of the emerging water occur at the bottom because of the absorption of energy from the surface. This is contrary to the normal flow conditions in a river where the velocity immediately above the bottom is a relative minimum. To prevent scouring, due to these high velocities (see Equation (4)), at the bottom, the river bed would have to be protected far beyond the lower end of the superficial eddy which, however, in most cases, would add to the cost in such a way as to make the structure economically impossible.

Guided by these considerations Professor Rehbock has invented a device called "Zahnschwelle",* or "Notched Sill" (Fig. 57), which is placed at the down-stream end of the apron for the sole purpose of establishing the natural flow conditions immediately below. As the average velocity at the down-stream end of the apron is smaller than the wave velocity, the kinetic energy

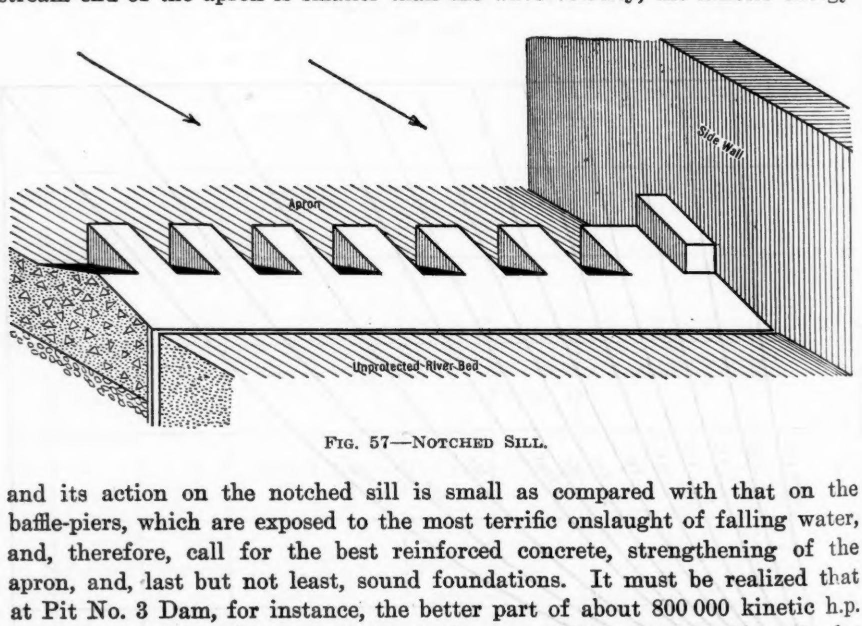

FIG. 57—NOTCHED SILL.

and its action on the notched sill is small as compared with that on the baffle-piers, which are exposed to the most terrific onslaught of falling water, and, therefore, call for the best reinforced concrete, strengthening of the apron, and, last but not least, sound foundations. It must be realized that at Pit No. 3 Dam, for instance, the better part of about 800 000 kinetic h.p. is "killed" within the zone of the baffle-piers. This undoubtedly calls for adequate structures, especially where concave surfaces are struck, lest the "killing" is done by the "horses" instead of the piers.

* U. S. Patent No. 1 561 796.

It is for these reasons and the fact that the notched sill has successfully stood the actual flood tests that the writer ventures to call it the best solution of the scouring problem, both from a scientific and an economic standpoint.

The principle of its action is that the rushing water sheets immediately above the apron are deflected in an upward direction by the vertical face of the teeth, thereby producing a long and flat ground eddy immediately below the sill which revolves as shown in Fig. 54; that is, its lowest sheets move in an up-stream direction. This movement is counteracted by the lower threads of fan-shaped jets pressed through the gaps between the teeth so as mutually to "destroy" energy without appreciable scouring of the unprotected river bed (deposition has been observed in most cases instead). The upper fan-shaped threads prevent the main stream, deflected upward by the teeth, from dropping down on the bottom which it would otherwise tend to do. Through the concentration of water threads near the surface, the discharge velocities there are increased. This, with the simultaneous decrease of the bottom velocities, leads to the normal distribution of velocities in the discharging stream.

Following the startling results in the laboratory, the notched sill was first installed on a weir of the Kraftwerk Friesland in East Prussia where, in the spring of 1924, a flood of considerable magnitude and a duration of 30 days went over the weir without causing any appreciable scouring, in spite of the fact that the river bed is mostly sand. The peak discharge exceeded 10 000 cu. ft. per sec., and the maximum head, 40 ft. Since then, many installations of the notched sill have followed with very satisfactory results, as far as the writer knows.

The height of the sill required is only about one-sixteenth to one-twenty-fourth of H (Fig. 54), whereas the height of the teeth varies between one-eighth and one-twelfth of H. For Pit No. 3 Dam a height of 4 ft. for the sill and 8 ft. for the teeth would probably be more than sufficient; whereas for Pit No. 4 Dam one-half the dimensions would do.

Furthermore, experiments have shown that, if a notched sill is used, the apron may advantageously be only two-thirds to three-fourths the length of the superficial eddy, or, in the present case, say, 120 to 130 ft. long for Pit No. 3 Dam and 100 to 116 ft. long for Pit No. 4 Dam, respectively.

The writer is indebted to the authors for their kind co-operation in furnishing plans necessary for the calculations involved.

MORROUGH P. O'BRIEN,* JUN. AM. SOC. C. E. (by letter).†—Although a few model experiments have been made in the United States, the prevailing opinion among American hydraulic engineers seems to have been, at least until very recently, that they are more a matter of academic interest than a basis for actual design. The present series of experiments are encouraging since they indicate that this attitude is changing. A recent German publication, "Die Wasserbaulaboratorien Europas", which describes the European hydraulic laboratories and their methods of study, shows that in those

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† Received by the Secretary, April 18, 1928.

countries where engineering knowledge is most advanced and economic pressure great, model experiments find their widest use as a guide to hydraulic design. In the introduction to this volume, Dr. de Thierry, one of the foremost hydraulic engineers in Germany, makes the statement that far from being an expensive luxury, laboratories for model studies are an absolute necessity for the economic design of hydraulic structures.

In the use of models for hydraulic studies, there are certain limitations which, if they are not known and properly taken into account, may cause the results to be grossly in error. The principles of dimensional analysis and dynamic similarity, on which the whole theory of models is based, cannot be applied blindly. In deriving these model laws, certain assumptions are made and, consequently, the conditions in the models must be carefully analyzed to see whether these assumptions are approximately realized. From an examination of many model experiments, the writer is of the opinion that a scale effect is to be expected in nearly every type of model, but that, by carefully regulating the conditions in the model, an experienced investigator can keep this scale effect or error within allowable limits.

In making model studies, the first step, and the most important one, is the proper choice of the model scale. This choice is dependent on many factors among which may be listed: Cost of model and of observations and computations; purpose of experiment and the accuracy desired; absolute dimensions of the model; velocities in the model; available materials for surfacing the model; ease of making observations; effect of surface tension and viscosity; effect of constant atmospheric pressure; and available measuring instruments.

In general, it is impossible to state that a certain scale ratio is satisfactory for model experiments. In some cases, a scale of 1:10 might give too small a model while in others a scale of 1:500 might give one that is too large. For experiments on some types of structures, such as bridge piers, where the formation of surface waves constitutes an important part of the phenomena, the velocities, and hence the vertical scale ratio, cannot be reduced below a certain point if size of the waves is to follow the scale of linear quantities. If there is a possibility of a reduction in pressure at important points in the model, the scale of the model is limited by the fact that the atmospheric pressure is the same for both model and prototype. In river models, where the depths and velocities are likely to be small, the manner of flow may be determined by the viscosity. The scale ratio chosen should be such that the cost of the experiment will not be excessive and that the results will be sufficiently reliable for the purpose at hand. Besides using geometrically similar models, it is possible to reduce the cost by using distorted models, vertical sections, and half-models. The studies of the Berak Power Plant, in India, were made with a complete model to a scale of 1:96 and with vertical sections through the sector gates, bottom outlets, and spillways to scales of 1:36 and 1:52. The hydraulic and economic requirements are always in opposition and the proper compromise between them can only be made after considerable experience in this kind of work.

The accuracy and reliability of experiments on models depend on the scale ratio and, of course, on the accuracy of observation. Studies at the Hydraulic Research Institute, at Munich, Germany, show a very good agreement, both as to the discharge and the erosion, between a model built to a scale of 1:50 and its prototype, one of the small dams of the Mittlere Isar Project. In these experiments the size of the model was reduced by making it a half-model which was allowable since the structure was symmetrical about the center line of the stream. A few years ago a model of the Harbor of Stockholm, Sweden, constructed to a horizontal scale of 1:300 and a vertical scale of 1:100, was tested at the Technical University of Stockholm, and it was found that the velocities indicated by the model agreed within 10% with those in the harbor. This divergence was decreased when the layer of slightly saline sea water in the harbor was reproduced in the model to the proper scale. While the turbines at Lille Edet were being tested, a rough check was made on the agreement between the actual conditions of flow and those shown by the model, which had been used in the design. It was only possible to examine the principal currents, but these were found to agree exactly.

The great savings in cost of construction that are very often made possible by model experiments are shown very clearly in the publication previously mentioned describing the European laboratories. As an additional instance may be cited the power plant of the Swedish Government at Trôllhättan where it was proposed to increase the head on the plant by constructing underground conduits from the tail-race to a point in the river below a controlling section. The proposed plan would have cost about 3 000 000 Swedish crowns. A model experiment was made at the Technical University and it was found that these conduits would not have the desired effect, due to peculiar eddies at the intake. As a result of the model studies it was decided to place much shorter conduits on the opposite bank of the river at a cost of 900 000 crowns. Full-scale experimentation would have wasted several million crowns.

In making experiments on river models, particularly on the dissipation of kinetic energy at the foot of a spillway, it is of the greatest importance to know the water level in the river bed below the dam for different stages of flow. If a controlling section occurs below the apron, it is necessary to increase the roughness of the model locally until the proper flow line is obtained. Otherwise, the depth of flow can usually be controlled by means of an adjustable weir.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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THE SCIENCE OF FOUNDATIONS—ITS PRESENT AND FUTURE

Discussion*

By Messrs. Lazarus White, R. D. N. Simham, and Frank A. Marston.

Lazarus White,† M. Am. Soc. C. E. (by letter).‡—Those of the Engineering Profession interested in foundations—and who is not—should count themselves fortunate that a man like Professor Terzaghi with his philosophical, mathematical, geological, and scientific equipment, has turned his attention to this heretofore obscure subject and has thrown great beams of light upon it, cutting like swords the numerous fallacies which have checked past progress.

When the writer was an undergraduate student he was thoroughly imbued with the classical methods of computing earth pressures, bearing values of soil, distribution of pressures, and pile-driving formula commonly taught—Rankine, Baker, Cain, and Wellington—and, after his graduation thirty years ago, he set about to apply them. Fortunately, he was also imbued with the scientific philosophy of Tyndall and Huxley—to accept no theories or formulas unless borne out by experiment. Huxley stated that one clear experiment not accounted for by the accepted theory might forever upset that theory.

The writer's first foundation experience, involving much wood pile-driving, had a disastrous effect on his faith in the *Engineering News* formula. No single experiment conformed to it and to "hit the target" of an observed result, was like shooting at a distant pin with a blunderbuss. Later, during subway construction, he observed that contractors completely and successfully ignored Rankine and Cain in timbering work, and "got away with it" in so many instances that by no scientific philosophy could their theories be justified. Again, his faith in Baker's "Masonry" was upset by the observation that

* Discussion of the paper by Charles Terzaghi, M. Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Pres., Spencer, White & Prentis, Inc., New York, N. Y.

‡ Received by the Secretary, April 9, 1928.

uniform loading did not produce uniform settlement. Hence, with practically all the classical notions upset, the writer decided to observe foundation phenomena for himself and to accumulate data bearing on a new science of foundations.

In 1914, during extensive underpinning operations along William Street, in New York City, many steel cylinders, 14 in. in diameter, were driven by hand hydraulic jacking into ground composed of varying mixtures of sand and clay. This afforded an opportunity to measure accurately the pressures applied. During the work the elastic properties of earth were observed, especially the rebound after releasing the load, and these were plotted by the engineers of the Public Service Commission of New York.* In 1915, Mr. J. A. Moyer and Professor Fehr, at Pennsylvania State College, published the results of tests† in the distribution of soil pressures measured at various depths and various eccentricities in relation to the applied loads. (See Fig. 25.) Using these observations as a basis, John F. Greathead, Assoc. M. Am.

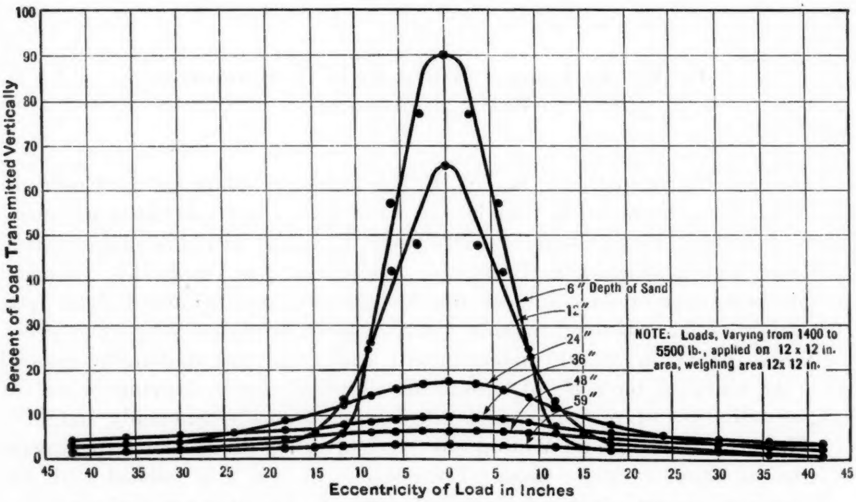

FIG. 25.—PERCENTAGE OF LOAD TRANSMITTED FOR DIFFERENT DEPTHS OF SAND.

Soc. C. E., Section Engineer in the days of the William Street Subway, plotted lines of equal vertical pressure beneath a 13½-in., circular plate,‡ disclosing in a manner which can be readily grasped the general distribution of pressure beneath a footing. This he called the "bulb of pressure"; so far as is known, it was the first time such a diagram was published. (See Fig. 26.) Due to the manner in which the experiments were made—the pressure being recorded on an area equal to the area originating the pressure—it does not show the variations in pressure immediately beneath the footing.

M. L. Enger, M. Am. Soc. C. E., published another bulb of pressure and, because of the use of smaller recording areas, was able to show variations in

* *Engineering News*, Vol. 85, No. 27, p. 1268, Fig. 2A.
† *Engineering Record*, Vol. 71, No. 11, March 13, 1915, p. 330.
‡ *Engineering News-Record*, December 30, 1927, Fig. 3, p. 1037.

pressure immediately below the footings. (See Fig. 27.) In general form his diagram is similar to that of Mr. Greathead, but it is more accurate.*

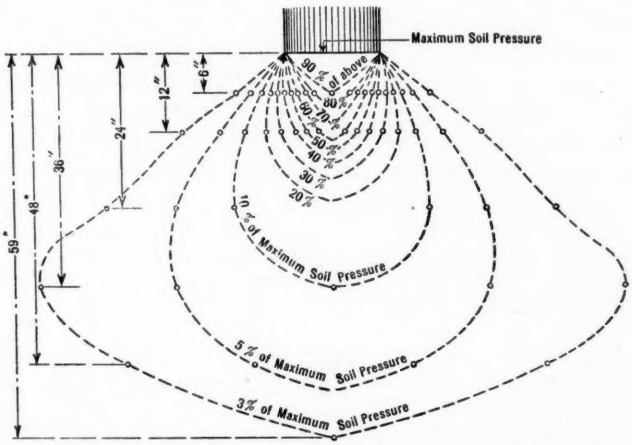

FIG. 26.—LOAD DISTRIBUTION BY BULB OF PRESSURE.

At the same time Professor Enger discovered the great variations in intensities of pressure under footings of different areas, as compared with average unit loads. His formula for such variations is $p = 91 \frac{d^{1.86}}{h^{1.95}}$, in which, p is the ratio (percentage of average unit load) at the depth, h, immediately below the center of the footing, and d is the diameter of the footing. This is of great interest, because if it is assumed that the soil has elastic properties and that a stress-strain relation exists (the applied loads giving the stress and the settlements, the strain or compression), the bearing value of large footings cannot be directly proportional to their areas.

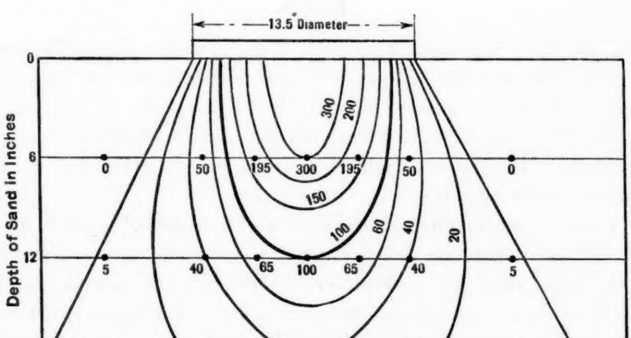

FIG. 27.—LINES OF EQUAL VERTICAL UNIT PRESSURE IN PERCENTAGE BASED ON 4-INCH PLUG TESTS.

Later, the Joint Committee on Stresses in Railroad Track of the Society and the American Railway Engineering Association in its report,† showed

* *Engineering Record*, Vol. 73, No. 4, January 22, 1916, p. 106.
† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1545.

the stresses in ballast below railroad ties as ascertained by an elaborate apparatus. Of special interest was a pressure capsule by which intensities of pressure at various points through a mass of ballast could be determined simultaneously. This report is a mine of information for the foundation engineer, especially the full demonstration of the elasticity of the soil beneath a railway track. A new "bulb of pressure" was published and—even more interesting—the effect of one footing on its neighbor. It was found to be the same as superimposing one "bulb of pressure" upon another or, rather, overlapping one with another and plotting their combined effect. (See Fig. 28.)

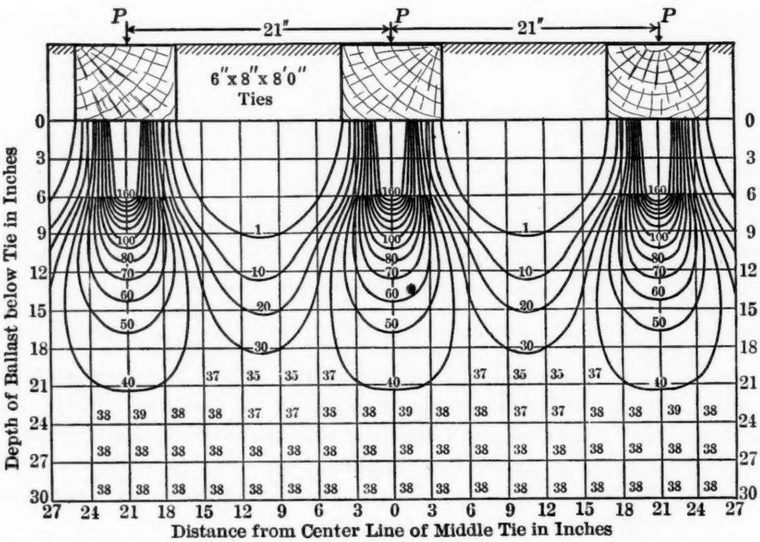

FIG. 28.

In 1925, A. T. Goldbeck, Assoc. M. Am. Soc. C. E., published* a curve indicating that the bearing values of footings or loads for equal settlement were not proportional to areas, but to diameters. Mr. Goldbeck invented a pneumatic pressure gauge, with electrical contacts,† which gave very accurate results. For bearing blocks of 0.371 to 8.33 sq. ft., the device checked the Enger formula quite closely.

In the writer's experience, buildings on compressible soil do not settle uniformly, but in parabolic or catenary curves; the center of the building settles much more than the perimeter. This is particularly marked in the City of Mexico where numerous structures show the characteristic curve in their courses. The reason for this should be apparent from data given herein. Since buildings exert pressure on huge areas the intensity of pressure at the center is very great, and they cause a corresponding compression of the ground, so that the building finally assumes a dished shape. (See Fig. 29.) The fallacy of the old assumption that structures, uniformly loaded, settle uni-

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 264.

† *Proceedings*, Am. Soc. for Testing Materials, Vol. XVI, p. 309, 1916.

FIG. 29.—VIEW OF SCHOOL OF MINES, CITY OF MEXICO, SHOWING DIFFERENTIAL SETTLEMENT OF ABOUT 3 FEET AT CENTER OF A BUILDING UNIFORMLY LOADED.

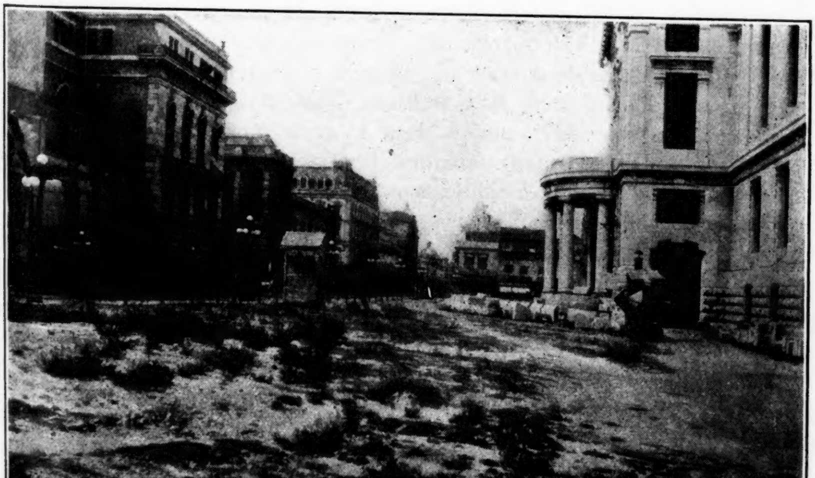

FIG. 30.—VIEW OF NATIONAL THEATRE, CITY OF MEXICO, SHOWING 6-FOOT SETTLEMENT OF BUILDING AND ENTIRE AREA AROUND IT.

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formly, is thus graphically shown. Fig. 30 shows the settlement of a building with the entire area around it. The original level of the building was the same as the sidewalk. The total settlement was approximately 6 ft.

The Society's Special Committee on Bearing Value of Soils for Foundations, etc., has performed its work painstakingly, but in a field very difficult and rather barren of results; that is, soil classifications and nomenclature to define soils. An infinite number of definitions can be made and yet one may be "left in the dark". Professor Terzaghi has taken foundation engineers "out of the woods" as far as nomenclature is concerned, and, by establishing a few simple physical classifications, it now seems possible to evaluate soils in terms of bearing power. This work is well begun by the U. S. Bureau of Public Roads under his direction, and more valuable results may be expected soon.

In describing* the characteristics of clay and its behavior in the presence of water due to viscosity and surface tension, he performed a valuable service. Recently, in Detroit, Mich., while excavating in clay and tapping wet areas under pressure, subsidence of surrounding buildings was observed which conformed to the principle laid down by Professor Terzaghi. A wet clay layer, in which water was supplied from seamy bed-rock below, when tapped at a depth of about 100 ft., yielded a considerable quantity of water. Simultaneously with the pumping of this water, settlements of neighboring buildings, several hundred feet away, were noted. Through the underlying rock the water beneath the subsiding area was partly drained—thus causing shrinkage in the wet clay layer above, which was equivalent to superimposing a load sufficient to squeeze the same volume of water out of the clay.

Another effect, noted at Detroit and Albany, N. Y., was that the wrecking or removal of buildings was followed by slight settlement of neighboring structures, where the region was underlaid by a stratum of wet clay.

It has been long observed that buildings on compressible soil do not settle uniformly as to time, but rapidly at first and then more and more slowly. This has been vaguely ascribed to the fact that water is squeezed from the saturated soil below; but by means of the "thermodynamic parallel", Professor Terzaghi has been able to evaluate this phenomenon and to plot a theoretical time-settlement curve of great value. This also has a geological application to alluvial deposits, and indicates that thousands of years may elapse before an alluvial deposit is consolidated. In this connection the geology of a stratum is of great significance in locating valuable structures. Nearly all old geological strata are fully consolidated and, therefore, have great bearing capacities, whereas recent alluvial deposits bordering streams, old ponds, or lake beds are dangerous. The latter are not fully consolidated, and their excess of water beyond that which is normal for the old strata is not yet squeezed out. Although the deposit may seem stable, an additional load will cause a flow of water and a subsidence or settlement. In some situations a trained geologist may be a better guide than the engineer, even if the latter has had much experience with foundations, and none in theoretical geology.

To illustrate how a structure settles, the information given in Figs. 31 and 32 is very interesting. Fig. 31 is the plan of a building on which settle-

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2286.

ments were observed between 1920 and 1924. Fig. 32 (*a*) to (*g*), inclusive, illustrates the catenary curves of walls resting on compressible soil. The various curves show that the time settlements are rapid at first and then become slower.

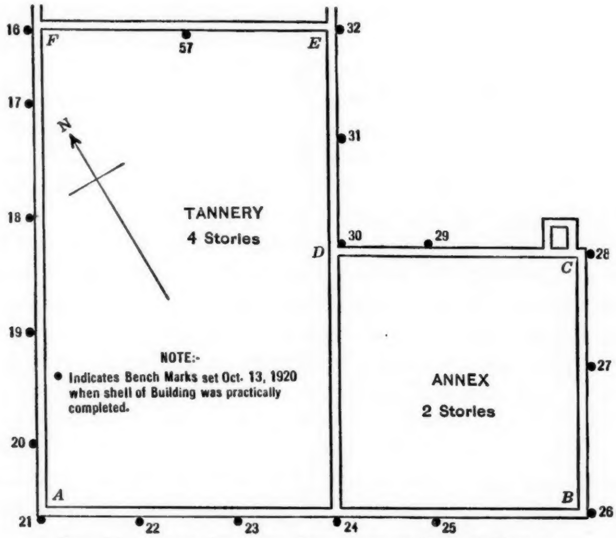

FIG. 31.—PLAN SHOWING LOCATION OF BENCH-MARKS.

The materials beneath the building were thoroughly explored. All test holes (see Fig. 32 (*b*)) were drilled 7 ft. south of the face of the south wall (*A-B*) and about 1 ft. inside the curb line. At Bench-Mark 20 (Fig. 32 (*b*)), a pipe was pushed to about 37 ft. Light driving sent it down to 41.7 ft. At this point the penetration was about $\frac{3}{4}$ in. per blow until, at a depth of 47.2 ft., the last twenty blows sent the pipe down $\frac{9}{16}$ in. and the driving was stopped.

At Bench-Mark 24 (Fig. 32 (*b*)), a pipe was pushed to 33.7 ft. before it was necessary to strike a blow. Then, for each successive increment of twenty blows, the penetration was as follows:

Total penetration, in feet.	Penetration for twenty blows, in inches.
35.5	6.5
40.7	4.5
46.1	3.5
46.9	2
48.0	1
48.29	0.19

After a total penetration of 48.29 ft., the next twenty blows failed to move the pipe. A 12-lb. sledge-hammer was used for driving.

The author has dealt the *Engineering News* formula "some staggering blows". Would that it had been "knocked out" completely. It was originated by Wellington, who really intended it to be applied only to uniform materials

like sand, with the drop-hammer then used. He introduced constants to give it a reasonable value. Theoretically, it has a factor of safety of 6, that is, a wooden pile which, by the *Engineering News* formula, gives a working value of 15 tons, has an ultimate value of 90 tons, which is absurd for a small wooden pile.

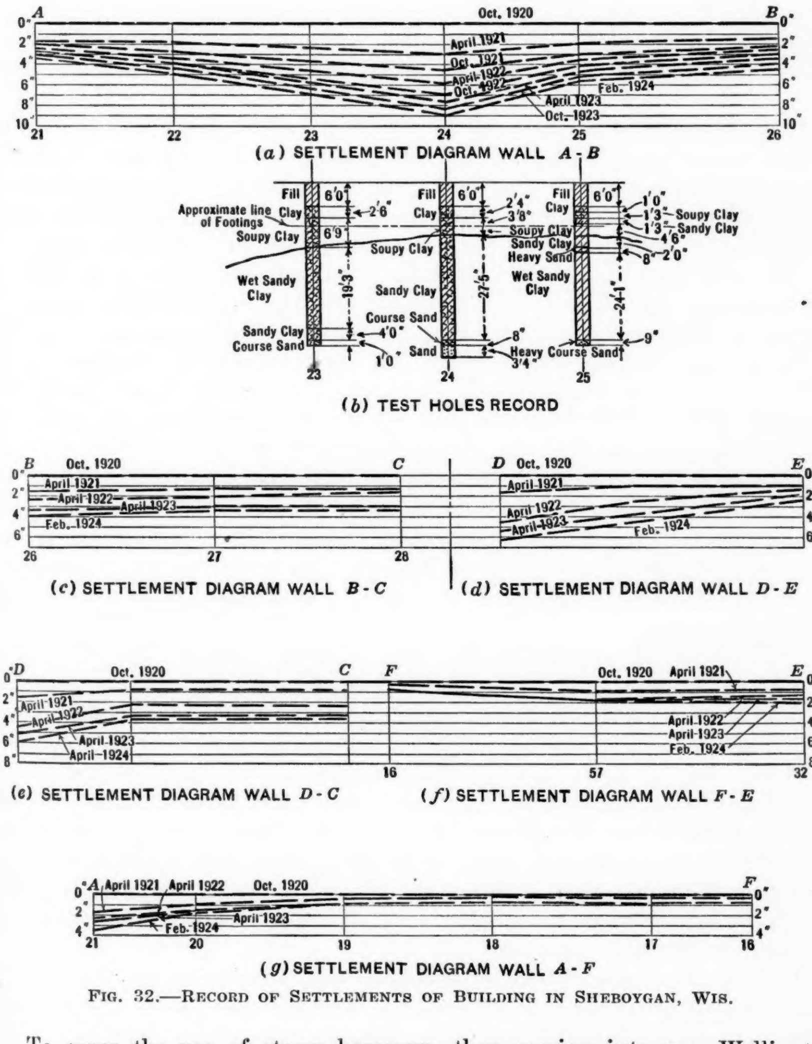

FIG. 32.—RECORD OF SETTLEMENTS OF BUILDING IN SHEBOYGAN, WIS.

To cover the use of steam hammers, then coming into use, Wellington arbitrarily changed his denominator, $S + 1$, to $S + 0.1$, giving no good reason except that the more rapid blow of the steam hammer ought to give that much better results. As a matter of fact, there is much variation in the rapidity with which steam-hammer blows are struck. The Vulcan hammer is nearly as slow as the drop-hammer and the double-acting hammer is much faster.

The *Engineering News* formula, now incorporated in many building codes, has done a great deal of harm, because its official character has misled owners and engineers as to its reliability. The writer has encountered cases where pile foundations, conscientiously driven to the *Engineering News* formula, have failed. He has repeatedly checked results obtained by using the formula, testing the driven piles with hydraulic apparatus, and has found the formulas decidedly inaccurate. The use of the *Engineering News* formula has checked progress. Because of its official adoption, engineers have often computed pile values by it and have not made tests which they would otherwise have made. In this connection it should be noted that the result of a loading test on a single pile should not be conclusive as to a group. Each pile of a rather closely spaced group will, on individual test, have a much higher value than the total of the group divided by the number of piles. This is because of overlapping of the areas supporting each pile—a similar effect which causes small areas to have larger bearing values in proportion than large areas.

The attempt to find a pile formula of general application is as futile as the search for the "philosopher's" stone. In this respect foundation engineers are worse off than they were before the invention of pile formulas when they relied on tests of piles driven under similar conditions to establish values. The only safe tests are those on a sufficiently large group to eliminate overlapping effects. Such tests will demonstrate the wastefulness of spacing of piles closely when not driven to rock, or an equivalent bearing.

Professer Terzaghi states* that piles are often driven in areas where they add nothing to the bearing value of the soil and sometimes even detract from it and that the utmost value that can be developed in such locations is that of an intelligently designed spread footing. The writer has even seen cases where driving piles in wet clay has had a negative value—because of the vibrations set up.

The author has discussed† the very important question of the relative bearing values of small and large spread footings and has thrown a great deal of light upon it, although there is much need of further investigation along these lines. That the bearing values of large footings are not in proportion to their areas as compared to small footings is to the writer, sufficiently proved. He has encountered many cases where settlements of large footings were much more than those for smaller ones. In a case where a high building was founded on a soft coral rock, the smaller footing settled about ½ in. and the larger, 6 in., although in this case the larger footings were designed with lighter unit loads than the smaller ones. Similar results were observed in buildings underlaid by peat, soft silt, etc.

In designing spread footings the old and easy assumption for the designer (that all that is necessary is to assume a working unit load—so many tons to the square foot—and then to divide this into the column loads to get the areas of spread footings) has led, and will continue to lead, to failures. The heavier the loads the more conspicuous the failures. Account must be taken of the shape of the footings, their relative sizes, their spacing, whether or

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2281.
† *Loc. cit.*, p. 2283, Fig. 10.

not they are too close together to work independently, the proportion of dead and live load, etc. On this point the writer begs to differ with Professor Terzaghi as to the value of loading tests. If intelligently made, they have great value.

Foundation engineers do not have all the data they could wish—not by any means; but they do have enough to avoid pitfalls which they have fallen into in the past and to produce designs far more intelligent than those heretofore considered good practice. Engineers may even decide not to build high buildings on certain areas with soils not adapted to carry heavy loads, and there are many such areas within the limits of large cities.

The author is rather pessimistic as to the value of soil tests as an aid to the proper designing of foundations. The writer, however, believes that soil tests, intelligently made and plotted, are of great value. The complete settlement curve should be obtained. The ideal apparatus for this is the hydraulic jack, with which reliable data for a complete curve can be rapidly obtained. This is illustrated in Fig. 33. It is much superior to the method* advocated by the Society's Special Committee on Bearing Value of Soils for Foundations, etc. By testing bearing areas of different sizes, a valuable relation may be found for use in designing spread footings. In the case of the building on coral rock, previously mentioned, bearing areas of 1 and 2 sq. ft. were tested, using pig iron for the loads. These tests, plotted as settlement curves, plainly revealed that the bearing value varied with the diameters of the footings, the area of 2 sq. ft. having only 1.4 times the value of that of 1 sq. ft. Had the designers been cognizant of this relation, the impracticability of any spread footing for this building would have been evident.

The results shown in the paper† in connection with sand-mica mixtures are most interesting but undue importance may be given to the shape of the grains. Is not the compressibility of the mixture high because the mica powder itself is very light and has a high percentage of voids? Its mixture with sand increases the percentage of voids very much. Any mixture with a large voids-ratio, or, in other words, with a low specific gravity, is bound to be compressible and unstable. The writer has observed natural sand deposits, apparently highly micaceous, which are dense and of good bearing value, although it may be that actual measurement would show the percentage of mica to be small.

The writer does not wish to disparage the use of sand-mica mixtures to simulate earthy materials with various voids-ratios, but wishes to express an opinion that the compressibility of soils is due more to lack of consolidation or to abnormally low specific gravity for the material rather than to the presence of mica or flat grains, although that is a factor in compressibility.

In the paper‡ the three properties that determine the behavior of the soil in the foundation pit are given as: First, the volume change produced by an increase of pressure acting on the soil, etc.; second, the permeability of the soil, etc.; and third, the cohesion or shearing resistance of the soil.

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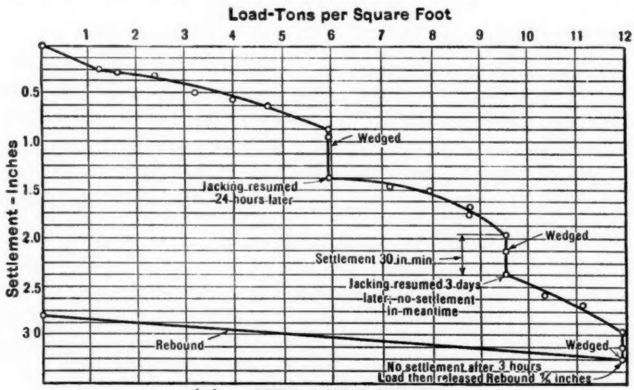
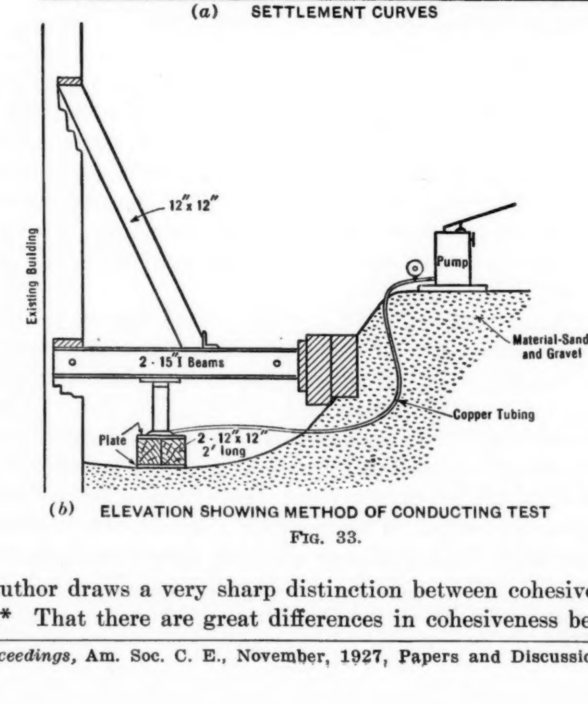
* *Proceedings*, Am. Soc. C. E., March, 1922, Papers and Discussions, p. 527.
† *Loc. cit.*, November, 1927, Papers and Discussions, p. 2290.
‡ *Loc. cit.*, p. 2289.

The writer would like to add a fourth, which may perhaps be a combination of the first two, but which is nevertheless important and easily ascertained; that is, the density of the soil or rock in relation to an old and well consolidated soil of similar nature. For instance, if the material is sand, determine its weight as compared to a sand of an old and well consolidated geological period, and if it is clay, find a similar relation. A very wet, compressible clay will necessarily be lighter because of its water content than a dry well consolidated clay of an old period; silt will be very light when compared to a consolidated earth, and, similarly, a coral rock will be very light compared to an old limestone. This relation is not hard to find, because old earths and rocks have a surprisingly uniform specific gravity.



(*a*) SETTLEMENT CURVES



(*b*) ELEVATION SHOWING METHOD OF CONDUCTING TEST

FIG. 33.

The author draws a very sharp distinction between cohesive and cohesionless soils.* That there are great differences in cohesiveness between different

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2269.

soils is not open to question, but in the writer's experience there is quite a residue of cohesion in all soils. Even clean sand such as that suitable for concrete will break in the bank with a curved fracture and stand vertical for a certain height. The writer is inclined to doubt the great effect of depth on settlement, unless the added excavation reveals a decided improvement of the ground. For instance, Professor Terzaghi states that, for a footing in perfectly cohesionless material, a depth of $\frac{t}{d} = 1$ reduced settlements to only one-third of what they would be if the footing rested on the surface of the ground.

Engineers are greatly indebted to the author for his discussion of the distribution of soil reaction over rigid loaded slabs.* He has shown clearly (for the first time to the writer's knowledge) the difference between the results as ordinarily computed with the common theory of uniform pressures and that of the much more correct theory of Professor Enger. A great difference is shown, the more correct theory yielding greater bending moments. This may explain why, in Detroit, a concrete mat supporting a large office building on clay is reported to show numerous cracks indicating failure. Single spread footings, such as the ordinary case of an individual footing for each column, the loads being concentrated under the center of application, will yield smaller bending moments than under the theory of uniform loading.

The writer feels with Professor Terzaghi that the future of foundation engineering as an applied science is decidedly encouraging and also that engineers are merely on the threshold of this science. The principle obstacles to progress having been removed, that is, various century-old assumptions that do not conform to Nature, the engineer can now go ahead.

Much can be added to the present knowledge of foundations by applying the methods originated by Professor Enger and Mr. Goldbeck for determining soil pressure. By placing pressure capsules in the soil beneath actual footings the distribution of stress can be determined so that it is possible to compute correctly the bending moments in footings. The relation of depth of footing to settlements and that of areas of footings to settlements, can also be determined, if not by this method then by direct loading tests on footings of different sizes and at different depths. By the use of hydraulic apparatus it is possible to obtain a wide range of tests in a limited time. By this method the effect of various shapes of footings on bearing capacities and settlements may also be found, and by testing groups of piles singly and in combination, the investigator can determine overlapping effects, etc., the proper spacing of piles for the best results, and the relation between the bearing capacities of a single pile and a group.

In the past engineers were content to state that a building settled very little, but accurate levels recorded, with comparatively little cost, would yield valuable and surprising data. In the future, various representative buildings should be chosen for the purpose of making accurate observations as to settlement. Professor Terzaghi is correct; foundation problems are of

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2269.

such a character that strictly theoretical mathematical treatment will always be impossible, but by systematically accumulating field data it will be possible to approach this ideal much more closely.

The writer hopes that the problem of soil classification will soon be solved along the lines suggested in the paper. He feels that the Engineering Profession is much indebted to Professor Terzaghi, first of all for uprooting certain persistent prejudices and then for leading engineers forward in new directions toward "the promised land" of a real science of foundations.

R. D. N. SIMHAM,* ASSOC. M. AM. SOC. C. E. (by letter).†—The writer desires to limit his observations on this interesting paper to the most important question of settlement of soils and its relation to the area of bearing and intensity of loading. It is a fallacy to generalize from experimental results without thoroughly understanding the premises on which, and the objects for which, the experiments are conducted. There are limitations to experimental researches that require corresponding limitations to be made in the laws deduced from them.

In the first place, any results that have been determined by individual experiments under limited considerations and very often in a slipshod manner, that is, without any attempt to follow a general or comprehensive plan and co-ordinated line or method of experimentation, cannot be complete evidence on which to establish any general or fundamental law of science. The writer has learned from experience that any attempts to deduce, from an unco-ordinated set of experiments, some sort of a general formula or law that would explain the entire phenomena of soil behavior, has invariably raised serious complications. Therefore, experiments can afford no relief to the present imperfect state of the science of foundations unless they are based on: (1) Constructive lines of thought; (2) a thorough realization of the assumed conditions; (3) a careful diagnosis of every factor that influences the behavior of a soil; and (4) a thorough co-ordination of all the information gained. "Knowledge comes, but wisdom lingers."

There is always a tendency to take some plausible results of experiments and immediately deduce laws of relationship between certain factors involved and then apply them to every single condition that occurs thereafter. Whenever inconsistencies are discovered, all kinds of sophistical arguments are made to explain matters, either by suggesting that the variations are exceptional, that they are unexpected, or are due to defective experiments, or other causes, but all the time insisting that the law, once derived by some mathematical manipulations of a set of convenient results, must apply to all sets of conditions.

In an ideal soil, uniform in every respect, it is possible to state that settlement is proportional to intensity of loading in some way and to the area in some other way. With regard to the area, the shape of the bearing surface has an influence on settlement; that is, the settlement of a foundation with the same load and bearing area, would be different for dissimilar shapes

* Town Planning Asst., Madras, India.
† Received by the Secretary, April 14, 1928.

of the bearing surface. Thus, in a simple way, the relationship between settlement of soil, s; load intensity, w; bearing area A; and the ratio of mean width to mean length of bearing section (or some other relation that would truly represent the effect of variation in shape and the consequent variation of volume of depth of soil affected), r, may be given in the general form:

$$s = \frac{w^x A^y r^z}{k} \qquad (12)$$

In this equation, k is a constant that expresses the characteristics of the soil.

If this may be taken as truly representing a law of behavior for given soils, then it would be easy to direct some experiments along definite lines and to arrive at the range between which the values of x, y, and z vary. However, in the determination of the value of k, the investigator would probably get into difficulties. The writer thinks that it is impossible to secure identical behavior of soils throughout the processes of varying load, area of bearing, and shape. Any settlement may be the result of several different reactions within the soil. There may be (1) a subsidence due to closer re-arrangement of materials of soil during a particular stage; (2) settlement due to plastic or flattening effect at some other stage, or combined with the former; (3) sinking due to escape or lateral flow of water or escape of other loose materials in the composition of soil; (4) shrinking due to compressibility of soil materials; and (5), under extraordinary circumstances, reduction caused by the crushing of the granules or disintegration of soil materials. Equation (12) may perhaps be modified further to cover these factors also,

$$s = \frac{w^x A^y r^z}{k - (c_1 + c_2 + c_3 + c_4)} \qquad (13)$$

In Equation (13), k may represent a value for soil under standard conditions, or a certain condition of soil behavior when settlement is a minimum. c_1, c_2, c_3, c_4, etc., may represent the reductions necessary in the values of k to allow for subsidence, settlement, shrinking, reduction, etc.

Any soil that is confined before loading to the limit of the natural angle of repose, XOA (Fig. 34), when loaded, slowly or rapidly adjusts itself in different angles until it attains the limiting angle of compactness of soil materials as indicated by the position, OB, in Fig. 34. Beyond this, a loading will cause the soil to flatten down to the limiting position of plasticity $C'C$. During this period, there may not have been any change in the volume of soil, but merely a bulging outward. Then sinking of soil occurs by the escape of water or loose material within the soil, until the limiting angle of permeability or soil escape, XOD, is reached. Further loading creates pure compression of the soil materials, and the soil shrinks to the limit of compressibility, $E'E$. Beyond this stage the soil sinks by the crushing of the granular shape. Thus, the actual curves of settlement *versus* load, based on experimental results would seem to illustrate, not one uniform law throughout, but different laws at different stages. Knowing exactly what is taking place within the soil itself, it should be possible to determine these laws independently through carefully planned experiments, and then, by an easy mathematical computation, a combined or general law of relationship might be deduced.

In another way, assuming that the total settlement of the soil is the combined effect of these factors, it may be found convenient to express the relation for determining the settlement in a somewhat better form, thus:

$$s = \frac{w^{x1}\,A^{y1}\,r^{z1}}{k_1} + \frac{w^{x2}\,A^{y2}\,r^{z2}}{k_2} + \frac{w^{x3}\,A^{y3}\,r^{z3}}{k_3} + \frac{w^{x4}\,A^{y4}\,r^{z4}}{k_4} \quad \ldots\ldots(14)$$

in which the quantities, $\frac{w^{x1}\,A^{y1}\,r^{z1}}{k_1}$, etc., represent the settlement due to the individual effects of each of the factors mentioned.

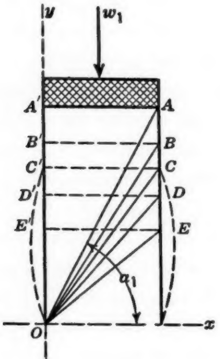


FIG. 34.

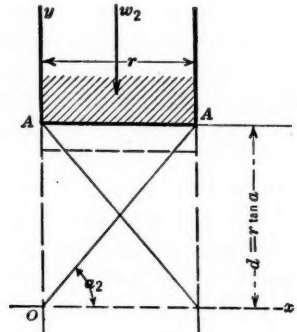


FIG. 35.

There are two more considerations which, instead of increasing the settlement, would reduce it. The side soil protection and frictional resistance offered by the marginal soil should modify the settlement of the soil directly below the foundation, and then Equations (13) and (14) will be further generalized to the following forms:

$$s = \frac{w^x\,A^y\,r^z}{k - (c_1 + c_2 + c_3 + c_4)} - \frac{V^m}{C} - \frac{P^n}{D} \quad \ldots\ldots\ldots\ldots(15)$$

and,

$$s = \frac{w^{x1}\,A^{y1}\,r^{z1}}{k_1} + \frac{w^{x2}\,A^{y2}\,r^{z2}}{k_2} + \frac{w^{x3}\,A^{y3}\,r^{z3}}{k_3} + \frac{w^{x4}\,A^{y4}\,r^{z4}}{k_4} - \frac{V^m}{C} - \frac{P^n}{D} \quad ..(16)$$

In Equations (15) and (16), V and P represent, respectively, the effective volume of soil contributing to the side protection of the compressed soil and the perimeter of bearing section; while m, n, C, and D are constants.

These do not necessarily comprise all conceivable factors affecting soil behavior. There is no doubt a difference in the behavior of soil under dead and live load and slowly or rapidly applied load. Eccentricity of loading and character and nature of distribution of thrusts on soil have also to be considered. In fact, it is necessary to know the law connected with each individual factor that influences the soil and then only can any comprehensive law or laws, which would correctly represent the soil behavior, be attempted. Any such attempt at the present state of knowledge on this subject will naturally be abortive. However, for practical purposes, it may perhaps be

sufficient to know what volume of soil will be affected by any load over any bearing area and deduce with reference to it, some simple relation for the settlement of soil of any specific composition under different loads.

The volume of soil that is directly affected by load is as shown in Fig. 35. This concept does not take into consideration, the soil that receives any indirect or virtual effects of load. It limits the volume to that portion lying constrained laterally within the vertical planes bounding the bearing area. The bottom would be defined by a horizontal plane (OX, Fig. 35) passing through the highest point of intersection of the plane (OA, Fig. 35) of shearing or cohesion and the vertical plane, OY. The direct settlement of the soil may be considered the result of (1) a closer re-arrangement of soil material; (2) actual compressibility; and (3) reduction due to permeability of this volume of soil or a large proportion of it. Any relation so determined will be useful in a practical way in studying comparative settlements or comparative bearing values of soil.

With these assumptions the relation between settlement and volume for any specific composition of soil may be simply written,

$$s = \frac{w^p \, (c \; V)^q}{k} \qquad \text{(17)}$$

In which, V is the volume defined in connection with Fig. 35, and p, c, q, and k are constants. In Fig. 35, let A = the area of the bearing surface; r, the mean or limiting width of soil that restricts the depth, d, of soil affected directly by the load; and a, the angle between the plane of shear or cohesion and a horizontal plane through O. Then,

$$V = A \; r \tan a \qquad \text{(18)}$$

Substituting this value of V in Equation (17),

$$s = \frac{w^p \, (c \; A \; r \tan a)^q}{k} \qquad \text{(19)}$$

For any specific soil c, $\tan a$, and k will be constant, so that settlement in such case would be represented by the relation,

$$s = \frac{w^p \, A^q \, r^q}{C} \qquad \text{(20)}$$

in which, C is a general constant for the characteristics of the specific soil.

As an example, select two kinds of soils to be compared for bearing values. Assume that s, p, c, A, r, and q remain the same in both cases; w_1 and w_2 are the supported loads; a_1 and a_2, the shearing angles; and k_1 and k_2, constants for the soil in each case. Then

$$\frac{w_1}{w_2} = \frac{k_1}{k_2} \left(\frac{\tan a_2}{\tan a_1}\right)^{\frac{q}{p}} \qquad \text{(21)}$$

Equation (21) explains in a simple way the values found by A. T. Goldbeck, Assoc. M. Am. Soc. C. E., with various mixtures of sand and clay.*

The author states that further efforts are being made under the auspices of the U. S. Bureau of Public Roads to determine certain simple routine

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 271.

tests. The writer hopes that the results of his research will be furnished to the Society so that it may be possible for the members, particularly those who reside abroad, to derive what benefit they can through such first-hand information.

FRANK A. MARSTON,* M. AM. SOC. C. E. (by letter).†—Considerable progress has been made by Professor Terzaghi in determining adequate and practical methods of classifying soils. For several years, the writer's firm has been carrying on laboratory and field studies on soils with the assistance of and by means of apparatus devised by the author.

In doing such work, there is a real benefit derived from the actual handling of the soil samples. It is not enough to have a trained laboratory man perform certain tests, but, in addition, some work should be done by the engineer responsible for the final decisions in order to acquire the "feel" of the various soils and to observe closely their characteristics as evidenced by varying behavior under test. While the external appearance of a soil sample should be studied, it may be most deceiving as to the actual characteristics and cannot be depended upon solely as a means of grouping soils.

As time goes on, methods and apparatus will be further simplified and their use will be more widespread. The great difficulty at present in the practical use of these improved methods of soil analysis is in the interpretation of the results. Only the gradual accumulation of data will provide the means for accurately translating laboratory results into terms of large scale field experience. In preparing papers such as this, Professor Terzaghi is rendering the profession a real service by arousing the interest of many investigators. If, as a result of these discussions and investigations, soil studies can be conducted according to standardized methods, records will be made that can be compared and used as a basis for judgment by engineers generally. A tremendous amount of data has been published regarding experiences with foundation conditions, but the lack of a suitable classification for the individual soils has made much of this data of little value.

An illustration can be cited showing the slowness of consolidation of the core material in a certain hydraulic-fill dam, with some data as to the character of the material according to these new lines of classification. All the material for this dam was obtained from a borrow-pit in shale located on a steep hillside which was covered with a layer of disintegrated material 3 to 4 ft. thick. This layer was the principal source of the core material.

Ten samples of the core material were taken at intervals from the top to a depth of 15 ft., at two points (Holes Nos. 1 and 2, Table 7), on the center line of the dam. These holes were 12 ft. and 150 ft., respectively, from the point of overflow of water from the pool during construction.

Samples of core material from the upper parts of the holes were obtained by pushing down a 3-in. pipe and withdrawing it filled with material. This method was continued until a depth was reached at which the core material was so soft that it would not remain in the pipe. Below this point, the holes

* Cons. Engr. (Metcalf & Eddy), Boston, Mass.

† Received by the Secretary, May 3, 1928.

were excavated by means of an earth auger which worked with fair success, except that some soft material slid off the auger as it was pulled up. The sampling tubes, 1 in. in diameter and 12 in. long, were used with some difficulty, due to the soft condition of the material which did not always fill the entire length of the tube.

When a ball of the core material was squeezed in the hand, it was found to be so plastic that it flowed out between the fingers. In one test hole, the material at a depth of 12 ft. contained so much water that, after standing over-night, the hole was found to be filled with water to the level of that in the reservoir. Table 7 gives the results of some of the tests of the core material.

There was little difference in the character of the samples from the two holes. The moisture in the samples as taken from Hole No. 1 varied from 19.3 to 42.4% and averaged 33.5% of the weight of dry solids. The samples from Hole No. 2 varied in moisture content from 26.4 to 48.2 and averaged 35.5 per cent. The moisture content did not vary uniformly from the top to the bottom of the holes. The average volume of voids in the samples from Holes Nos. 1 and 2 was 48.6 and 48.0%, respectively, computed on the volume of the wet material as sampled. The specific gravity of the dry core material was 2.748.

A comparison of the liquid limit given in Column (5), Table 7, with the moisture content given in Column (11) (both being expressed in the same terms), shows that about one-half the samples contained more moisture when taken out than at the lower liquid limit.

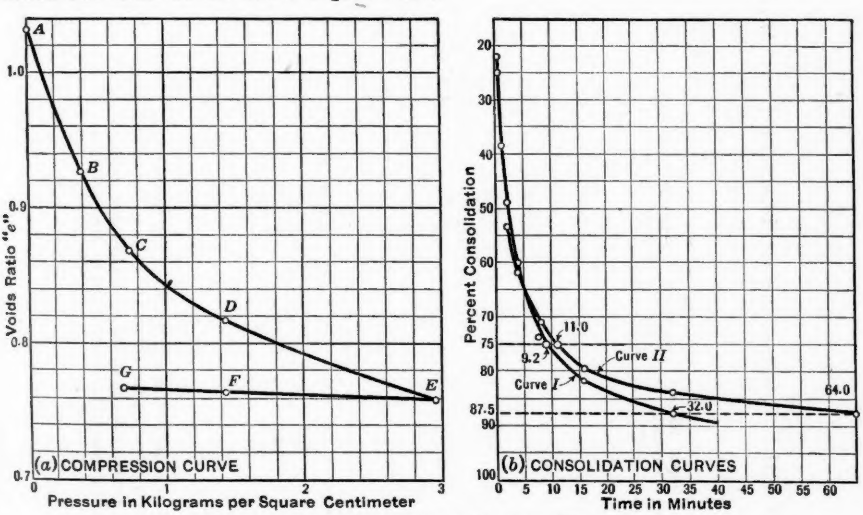


FIG. 36.—SAMPLE NO. 712 FROM TEST HOLE NO. 2.

A compression test was made on Sample 712 of this core material for the purpose of determining the compressibility and permeability of the material (see Fig. 36 and Table 7).

The dry material was made into a paste and placed in the cylindrical container of the compression apparatus in such a manner as to exclude air

TABLE 7.—RESULTS OF TESTS OF CORE MATERIAL FROM AN HYDRAULIC-FILL DAM.

Sample No.	Depth below surface, in feet.	Test Hole No.	Specific gravity.	LOWER LIMITS IN PERCENTAGE.			VOIDS RATIO AT LIMITS.†			Moisture content of sample as removed, in percentage of weight of dry solids.
				Liquid.*	Plastic.*	Difference.	Liquid.	Plastic.	Difference.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
603	....	....	2.778‡	33.5	23.1	10.4	0.931	0.642	0.289	....
(Old dry sample)	....	....	......	....	....	....	.....	.....	.....	....
703	3.0	....	2.770§	26.3	23.3	3.0	0.738	0.645	0.093	19.3
704	6.0	....	2.770§	33.2	23.4	9.8	0.917	0.648	0.269	31.3
705	9.0	1	2.770§	32.5	22.0	10.5	0.900	0.609	0.291	34.3
706	12.0	....	2.770§	37.3	28.7	8.6	1.034	0.795	0.239	42.5
707	15.0	....	2.770§	41.0	27.8	13.2	1.135	0.770	0.365	40.2
Average (703-707)	9.0	....	......	34.06	25.04	9.02	0.945	0.693	0.252	33.5
708	3.0	....	2.770§	42.2	31.5	10.7	1.078	0.872	0.206	48.2
709	6.0	....	2.770§	28.2	20.9	7.3	0.779	0.579	0.200	28.6
710	9.0	2	2.770§	32.3	23.2	9.1	0.895	0.642	0.253	40.5
711	12.0	....	2.770§	31.9	23.5	8.4	0.884	0.651	0.233	26.4
712	15.0	....	2.748‡	35.7	25.2	10.5	0.982	0.693	0.289	34.0
Average (708-712)	9.0	....	......	34.06	24.86	9.20	0.924	0.687	0.237	35.5

* Weight of moisture in percentage of weight of dry solids.
† The voids ratio is the ratio of the volume of voids to the volume of solids.
‡ Determined and used in computations relating to this sample.
§ Assumed and used in computations relating to these samples.

and completely fill the space. The paste was approximately at the liquid limit. Pressure was then applied at 0.39 kg. per sq. cm. Observations were made, by means of Ames dials, of the decrease in thickness of the specimen at frequent time intervals until there was no further consolidation. The load was then increased immediately to 0.74 kg. per sq. cm. and similar observations of thickness were made. Similarly, the test was repeated for loads of 1.44 and 2.96 kg. per sq. cm.

When consolidation had been completed, the load was reduced by the same increments by which it had been increased, allowing sufficient time for the sample to expand or rebound to a constant volume between changes in load. The moisture content of the sample was determined before and after testing. Other values used in the analysis were: Specific gravity, 2.748; plastic limit, 25.2 (e = 0.693); liquid limit, 35.7 (e = 0.982); diameter of the sample, 7.00 cm.; and reduced thickness, 0.705 cm. In Fig. 36 (*b*), Curve I is the consolidation curve when the pressure was increased from 0.74 to 1.44 kg. per sq. cm., and Curve II is the consolidation curve when the pressure was increased from 1.44 to 2.96 kg. per sq. cm.

Point *A* (Fig. 36 (*a*)) represents the voids ratio at the beginning of the test; Points *B, C, D,* and *E* represent the voids ratios of the completely consolidated specimen under the loads of 0.39, 0.74, 1.44, and 2.96 kg. per sq. cm. Points *F* and *G* represent the voids ratios when the specimen had rebounded after removing loads of 1.52 and 0.70 kg. per sq. cm., respectively. These observations indicate that the material consolidates almost as slowly as a highly colloidal clay.

The coefficients of consolidation, compressibility, and permeability were computed (see Table 8), together with similar figures for a fat plastic clay and a fine sand having an effective size of 0.1 mm.

TABLE 8.—COEFFICIENTS OF CONSOLIDATION, COMPRESSIBILITY, AND PERMEABILITY.

	COEFFICIENTS OF.		
	Consolidation.	Compressibility.	Permeability.
Fat Plastic Clay:			
Low pressure	1.72×10^{-3}	4.4×10^{-4}	7.6×10^{-7}
Medium pressure	2.62×10^{-3}	2.3×10^{-4}	6.0×10^{-7}
High pressure up to 2.96 kg. per sq. cm	2.10×10^{-3}	1.1×10^{-4}	2.3×10^{-7}
Core Sample No. 712:			
Low pressure	2.78×10^{-3}	1.69×10^{-4}	4.7×10^{-7}
Medium pressure	11.7×10^{-3}	0.74×10^{-4}	8.7×10^{-7}
High pressure up to 2.96 kg. per sq. cm	5.87×10^{-3}	0.38×10^{-4}	2.2×10^{-7}
Fine Sand, Effective Size = 0.1 mm.:			
Low pressure	$171\,000 \times 10^{-3}$	0.5×10^{-4}	$85\,300 \times 10^{-7}$
Medium pressure	$378\,000 \times 10^{-3}$	0.22×10^{-4}	$83\,300 \times 10^{-7}$
High pressure up to 2.96 kg. per sq. cm	$500\,000 \times 10^{-3}$	0.16×10^{-4}	$80\,000 \times 10^{-7}$

It will be seen that the coefficients of consolidation and permeability for the core material are similar to those of the fat plastic clay and vary greatly from those of the fine sand. The greater the coefficient of consolidation, the

more rapidly consolidation will take place. For the core material, the rate of consolidation is exceedingly slow. The indications are that the core in the dam built with this material to a depth of 15 ft. at least, is in about the same state of consolidation as it was when the dam was built, eight years before the samples were taken.

The influence of grain size on the variation in lower plastic and liquid limits and the difference between these limits is illustrated by data given in Table 9. These data were obtained by separating into its respective portions (according to grain size) a sample from the borrow-pit from which the core material was obtained. This material is a bluish or greenish shale which breaks into bulky pieces.

TABLE 9.—Lower Liquid and Plastic Limits of the Several Fractions of Fine Material from Borrow-Pit.

Size, in millimeters.	Proportion of total weight, percentage.	Liquid limit.	Plastic limit.	Difference.
Composite sample	100	33.0	24.7	8.3
0.1 — 0.02	35.2	26.9	29.8	0
0.02 — 0.006	22.2	38.2	36.8	1.4
0.006 — 0.002	17.2	42.2	29.6	12.6
Less than 0.002	25.4	66.8	36.1	30.7

For material between 0.1 and 0.02 mm. in size, the liquid and plastic limits should be practically identical. The discrepancy shown in Table 9 is due to the difficulty in determining accurately the plastic limit in such material. The effective size of the fine material was 0.00075 mm., and the uniformity coefficient was 20.6. The coarsest fraction, 0.1 — 0.02 mm., had relatively low liquid and plastic limits and the difference between them should have been practically zero. The very fine material (less than 0.002 mm.) had high liquid and plastic limits and there was a wide difference between them.

An examination of the borrow-pit material before construction by the methods indicated would have made it possible to forecast the behavior of the material.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

FLOOD CONTROL WITH SPECIAL REFERENCE TO THE MISSISSIPPI RIVER

A SYMPOSIUM

Discussion*

By Messrs. S. L. Moyer, William M. Hall, C. E. Ramser, and T. Kennard Thomson.

S. L. Moyer,† Esq. (by letter).‡—The wide divergence in run-off behaviors under the current conceptions of the relation of run-off to rainfall, is most effectively disclosed by an examination of the diagrams presented in connection with the paper by Messrs. Woodward and Nagler.§ This fact seems to indicate a very real need for a rational evaluation of the accidental influences which give rise to such a haphazard showing. This apparent need affords a timely opportunity to present an entirely different conception of the nature of the problem in hand.

The method of attack herein outlined assumes that there is some general law governing the accidental occurrence of storms, and that the chance for such events arises out of basic causes in combination, after the fashion of the "hit-or-miss" fall of the dice or cards.

As distinguished from the current theory of probabilities, this development does not assume that rainfall magnitudes are the result solely of causes in summation; but, on the contrary, it is conceived that these events may also be influenced by factors (dice of destiny) combining by multiplication, rather than summation, of values.

In common with other probability methods, time and count are herein assumed to be one and the same thing; and time in its totality or eternity

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* Discussion of the Symposium on Flood Control with Special Reference to the Mississippi River, continued from May, 1928, *Proceedings*.

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‡ Received by the Secretary, February 27, 1928.

§ *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 165.

is taken as unity or 100 per cent. The extent of time, or count, is measured in relation to the opportunity offered for the occurrence of magnitudes, without regard to the sequence of events; and the procession of time is viewed as to its content in event magnitudes, instead of in the ordinary terms of the chronological order of occurrence.

The words, expectancy and probability, are perhaps unfortunate in that they seem to suggest an element of uncertainty; but an expectancy examination of data, in its recognition of the accidental elements and in its attempt to assay the chance, elects to face a situation which other methods ignore. Once the fate factors are given proper weight, a surprising uniformity of results may be achieved.

An expectancy analysis was made, based on the data given for monthly rainfalls and run-offs for April, May, and June, during the two periods before and after drainage, covering the basins of the Des Moines River above Keosauqua and of the Iowa River above Iowa City. This single showing cannot be said to prove conclusively the validity of the approach to the problem in question. However, the apparent harmony of the results obtained from this analysis does seem to tend to prove that, on the expectancy average: (1) run-off is a proportion of rainfall after deducting a constant loss factor; (2) the constant loss factor is little, if at all, affected by agricultural drainage; and (3) as applying to the area actually subjected to drainage improvements, agricultural drains of the Iowa type have apparently effected a reduction in the run-off proportion to the extent that the value after drainage is about 40 to 45% of the original value before drainage.

An ordinary rain-gauge has a catchment area of (roughly) one hundred-millionth part of a square mile. In the case of the Des Moines and Iowa River basins, on which Weather Stations are fairly numerous, there seems to be about one significant rainfall record to a section of 300 sq. miles, or the gauge sample of the rainfall is about one thirty-billionth part of the section it presumes to represent. The gross inadequacy of such representation as evidence of what happens in a given storm on the 300-sq. mile section, may perhaps best be seen by comparison with a car of wheat.

An ordinary freight car transports about 1 500 bushels, or 90 000 lb. of wheat (1 440 000 oz.). An ounce of wheat contains somewhat less than 2 000 kernels; so a carload of wheat is made up of a scant 3 000 000 000 kernels. Assuming a sample of the wheat in proportion to the rainfalls in question, 1 kernel must be taken as a fair sample of 10 or 12 carloads, if the rainfall samples are fair, and the absurdity of the representation becomes self-evident.

On the other hand, it is a matter of common observation that different storms travel in different ways, so that during the course of a few years, the rainfall record at the gauge will include practically all the variations in intensity that fall to the lot of the whole section which the gauge represents. Therefore, the gauge record does fairly represent the trend of magnitudes for the whole section, and since expectancy has to do with the trend of magnitudes, the sample is fair for expectancy purposes.

It is obvious that run-off records fairly represent the whole of the events, either for a given storm or for the trend of storm magnitudes. The limita-

tions of mathematics demand that the association of values in any given operation, be confined to subjects and predicates displaying uniformity in logical kind or type. This restriction, which is the result of extensive mathematical experience, therefore, suggests that the only fair basis for the relation of run-off to rainfall, is found in that aspect under which rainfall samples appear to be fair, or in the expectancy viewpoint.

In its essence, expectancy or probability is an assay of time as to its content in magnitudes. The writer conceives time to be the opportunity for the occurrence of events. An event may be said to be a manifestation of activity, a behavior in the material, or a phase of transition, which occurs, happens, befalls, takes place, or comes to pass in a series, a parade, or procession of incidents.

The monthly rainfalls and run-offs for the Des Moines and Iowa River basins above Keosauqua and Iowa City, given in Table 31,* under the expectancy conception, may be regarded as events of the same kind or type with respect to the time lapse in which the magnitudes have accumulated. Some of the phenomena, however, are subject to widely different types of seasonal influence, and, therefore, may be said to be unfair in association. For this reason, all the magnitudes in Table 31 have been discarded, except those covering the months of April, May, and June in each of the years listed.

While these three months are not exactly of uniform seasonal kind or type, any difference is progressive and may be fairly regarded as reducible to expectancy. The inclusion of more than one month for each year of the record is not valid as to expectancy over a period of years, but this sort of representation is believed to result in assemblies of rainfall and run-off data which are logically comparable.

Even by this means of expanding the expectancy series by inclusion of several months in a given year, the remainder of the data given by the authors is entirely too scant for analysis by this method, so it must also be disregarded. Maximum 24-hour magnitudes for each of these months might be compared, but the writer prefers to have these data compiled by one who is a stranger to this system of analysis, in order to avoid any suspicion of "colored" statistics.

With fair samples on hand, observations to be used for expectancy purposes may next be graded, as sizes are screened and sorted in sand and gravel analyses. The accidental occurrence of the sizes in a gravel deposit, as to arrangement in place, is important in so far as it affects the picking of a fair sample, but after such sample has been secured, the original order of deposit has no particular significance. Similarly, the sequence of magnitudes in the time parade is of no consequence, after the expectancy samples have been selected, and the grading of the samples is effected simply by arrangement of the observations in the order of their sizes.

For the expectancy assay of events, no division into arbitrary classes, as in gravel analyses, is necessary because the analysis of the magnitude content of time is accomplished through the ratio of the count of observations,

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 171.

found above and below any given stage, or by the odds of the number of items of data exceeding and short of the position in question.

Every line of attack on the probability problem, including even the classic Gaussian law of error, attempts to evaluate the content of time in its eternity; and as this can never be exactly known, each such effort to solve the mystery must of necessity be merely an empirical approximation of magnitude trends. It is the writer's belief that the method herein presented, represents a definitely practical step in advance, affording a broader insight into the various aspects of the subject than any hitherto proposed. The broad field to which the idea may be applied, if it proves sound, warrants searching and critical study.

As the mathematical mechanism is an empirical development, the practical operation will be shown before attempting to state the logical reasoning that may be said to support the writer's analytical expectancy charts (Figs. 48 to 52).

Notation.—

Let F = the frequency, or the average interval in the count of events between magnitudes exceeding any given stage.

P = the probability, or proportion of the count of events exceeding any given stage of magnitude.

N = the numerical designation of an observation when the expectancy series is arranged and observations are numbered in the order of magnitude, beginning with the least.

T = the total number of observations in the expectancy series.

d = any position on an arithmetical base scale covering a range from zero to unity, or covering time in its totality.

v and z = two arbitrary variables influencing the form and position of the scale of F (Fig. 48) in reference to the basic and fixed scale of d, and by means of which data trends may be reduced to an approximate straight line for one-half or more of the range of the trend in a given approximation.

e = the tangent of slope for any straight line on a chart (Fig. 48) having the d scale as a horizontal base and magnitudes as vertical arithmetical ordinates; or the difference in magnitude on the straight line between the two limits of the d scale.

c = the ratio of the magnitude on the straight line at the upper limit to e; or that position on the d scale where the straight line intersects zero magnitude or would intersect zero magnitude if the d scale were extended.

Q = the magnitude for any point on the straight line.

A = the proportion of catchment area unaffected by drainage improvements.

B = the run-off proportion for the area, A, after deducting constant loss factor.

C = the proportion of catchment area affected by drainage.

D = the run-off proportion for the area, C, after constant loss deduction.

E = the run-off proportion for entire area after drainage, the constant loss factor being first deducted.

For the purpose of assigning logical frequency values to the various observations in an expectancy series of data, as a guide to the plotting of the magnitudes on the writer's analytical expectancy chart, it can be demon-

strated that Equation (1) produces values that are consistent, on the average, for both long and short series:

$$F = \frac{T + 2\left(\frac{T-1}{T+5}\right) - 1}{T + \left(\frac{T-1}{T+5}\right) - N} \qquad (1)$$

Expressing the numerator of Equation (1) by a, and assuming,

$$b = T + \frac{T-1}{T+5} \qquad (2)$$

Equation (1) may then be written,

$$F = \frac{a}{b - N} \qquad (3)$$

or, since P is the inverse of F,

$$P = \frac{b - N}{a} \qquad (4)$$

Equation (4) is not essential to the distribution of data in this analysis, but it is introduced in order that a clear conception may be had of the relation to the ordinary, current conceptions of probabilities.

As a general equation by means of which the law of chance curvature for occurrences of a given kind or type the following empirical expression is offered:

$$d = \frac{v}{F} + \frac{(1 - v)\ (1 + z)}{F + z} \qquad (5)$$

The form of Equation (5) is such that when $F = 1$, $d = 1$, and when $F = \infty$, $d = 0$. The limits of the d scale, therefore, always include infinite time or eternity. Writing Equation (5) in a form expressing the trend of rainfall and its consequences as to magnitudes expected,

$$d = \frac{3}{10\,F} + \frac{35}{F + 49} \qquad (6)$$

Equation (6) also approximates the curve form for four chance elements falling in accidental combination and influencing event magnitudes by multiplication of cause values.

The magnitude for any point on the straight line shown in Fig. 48 is determined by the formula,

$$Q = (c - d)\ e \qquad (7)$$

The writer's expectancy charts for rainfall and run-off (Figs. 48 to 52) are designed under and portray the law of chance expressed by Equation (6). By means of Equation (3), a series of observations of phenomena which are truly representative of the chance, may be assigned a logical frequency on the diagram against the F scale and plotted in proper frequency position. The graphic representation of the magnitudes may then be approximated by a straight line, and an equation of the expectancy representing the straight-line reconcilement of the data, may be derived under the principles of Equation (7).

In Table 39, the rainfall and run-off magnitudes listed in Table 31 and covering the Des Moines River above Keosauqua and the Iowa River above Iowa City during the months of April, May, and June, for which both rainfall and run-off magnitudes are listed, are arranged in the order of their respective sizes. These magnitudes have then been numbered in the same order and the frequency, F, has been computed as shown by the F values in Table 39, utilizing the principles of Equation (3).

TABLE 39.—FREQUENCY AND DATA.

Des Moines River, Keosauqua, Iowa.						Iowa River, Iowa City, Iowa.					
Before Drainage.											
N.	*F.*	Month.	Rainfall, in inches.	Month.	Run-off, in inches.	*N.*	*F.*	Month.	Rainfall, in inches.	Month.	Run-off, in inches.
1	1.07	April, 1906	2.40	May, 1906	0.52	1	1.07	April, 1906	2.29	May, 1904	0.58
2	1.235	May, 1906	3.18	May, 1904	0.61	2	1.235	June, 1903	2.45	May, 1906	0.64
3	1.458	May, 1904	3.22	April, 1905	0.66	3	1.458	April, 1905	2.91	April, 1905	0.70
4	1.778	April, 1905	3.42	June. 1905	1.00	4	1.778	May, 1906	3.03	April, 1904	0.82
5	2.28	June, 1903	3.63	May, 1905	1.04	5	2.28	April, 1904	3.54	June, 1905	1.03
6	3.18	April, 1904	4.37	April, 1906	1.14	6	3.18	May, 1904	3.69	April, 1906	1.69
7	5.25	June, 1905	5.66	April, 1904	1.25	7	5.25	June, 1905	4.59	May, 1905	1.73
8	15.	May, 1905	5.76	June, 1903	2.69	8	15.	May, 1905	7.12	June, 1903	2.59
After Drainage.											
(Area, 32.7% Drained).						(Area, 46.5% Drained).					
N.	*F.*	Month.	Rainfall, in inches.	Month.	Run-off, in inches.	*N.*	*F.*	Month.	Rainfall, in inches.	Month.	Run-off, in inches.
1	1.05	April, 1918	1.76	April, 1918	0.10	1	1.05	April, 1923	1.60	April, 1918	0.23
2	1.133	April, 1923	1.88	May, 1918	0.31	2	1.133	May, 1923	2.48	May, 1923	0.28
3	1.23	May, 1919	2.62	May, 1923	0.31	3	1.23	April, 1918	2.70	May, 1922	0.58
4	1.345	May, 1920	2.67	April, 1923	0.48	4	1.345	May, 1919	3.07	April, 1921	0.63
5	1.484	April, 1921	2.84	April, 1921	0.57	5	1.484	April, 1922	3.09	May, 1921	0.73
6	1.655	May, 1923	3.01	May, 1922	0.57	6	1.655	May, 1920	3.65	April, 1922	1.20
7	1.868	April, 1922	3.20	May, 1921	0.61	7	1.868	April, 1921	4.11	April, 1923	1.26
8	2.149	May, 1922	3.57	April, 1922	1.00	8	2.149	May, 1922	4.11	May, 1918	1.30
9	2.526	April, 1920	4.31	April, 1920	1.24	9	2.526	May, 1921	4.31	April, 1920	1.50
10	3.065	May, 1921	4.89	May, 1920	1.27	10	3.065	April, 1920	4.31	May, 1920	1.54
11	3.90	June, 1918	5.08	June, 1918	1.35	11	3.90	April, 1919	5.14	April, 1919	1.70
12	5.35	April, 1919	5.14	May, 1919	1.35	12	5.35	June, 1919	6.43	June, 1919	1.74
13	8.53	May, 1918	6.41	April, 1919	1.45	13	8.53	June. 1918	7.53	May, 1919	1.83
14	21.	June, 1919	7.05	June, 1919	1.67	14	21.	May, 1918	8.32	June, 1918	3.90

Based on the magnitudes and frequency values of Table 39, graphic distribution and representation of the data have been accomplished as shown in Figs. 48 to 51, inclusive.

Referring to Fig. 48, after plotting the data in accordance with the computations of frequency, these magnitudes have been approximated by a straight line for both the rainfall and run-off data. From the scales of this diagram,

values of the factors in Equation (7) have been derived as shown in Column (1) of Table 40. This process has been repeated for Figs. 49, 50, and 51, and the values derived as shown in the other columns of Table 40.

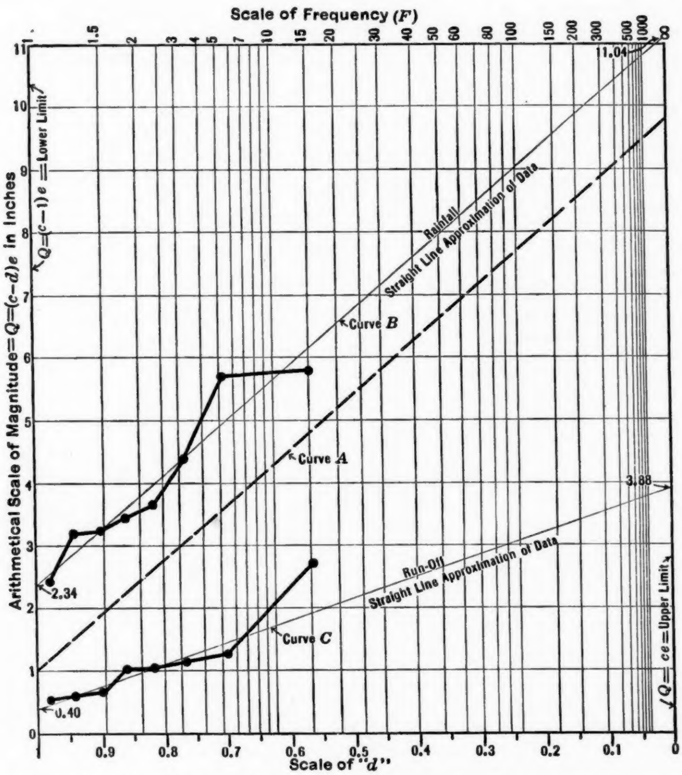

FIG. 48.—DES MOINES RIVER, KEOSAUQUA, IOWA, BEFORE DRAINAGE.

The approximation of the observed magnitude by a straight line in each of these cases is somewhat a matter of judgment, but from the very nature of the data arrangement, erratic magnitudes must tend to fall in the main in the upper or lower portions of the series. The magnitudes in the middle section are thus seen to be entitled to the greatest weight in such graphic reconcilement. In all eight of the series thus reconciled, no adjustments whatever have been made subsequent to the final computation of results.

In Table 40 (Column (1)), beside the item "Constant Deduction", appears the value, 1.34. In Fig. 48, a broken straight line (Curve A) is drawn parallel to and at a magnitude of 1.34 below the straight line for rainfall (Curve B). The value 1.34 is such that Curve A will intersect the straight line for run-off (Curve C), when both are extended, at the zero position on the scale of magnitudes. From this, it will be seen that after the deduction of 1.34 in. from rainfall, on the expectancy average, the remainder is related to run-off by proportion.

TABLE 40.

	Des Moines River.		Iowa River.	
	Before drainage, Fig. 48.	After drainage, Fig. 49.	Before drainage, Fig. 50.	After drainage, Fig. 51.
Rainfall, Straight Line:				
Q for upper limit	11.04	14.27	10.33	15.74
Q for lower limit	2.34	1.59	1.96	1.94
Value of e	8.70	12.68	8.37	13.80
Value of c	1.269	1.125	1.233	1.141
Run-off, Straight Line:				
Q for upper limit	3.88	4.22	5.72	6.71
Q for lower limit	0.40	0.09	0.22	0.12
Value of e	3.48	4.13	5.50	6.59
Value of c	1.115	1.022	1.040	1.019
Difference in c for rainfall and run-off, straight lines	0.154	0.103	0.193	0.122
$c \times e$, or constant deduction	1.34	1.31	1.62	1.68
Run-off proportion or ratio of e values, run-off to rainfall	0.400	0.326	0.657	0.477

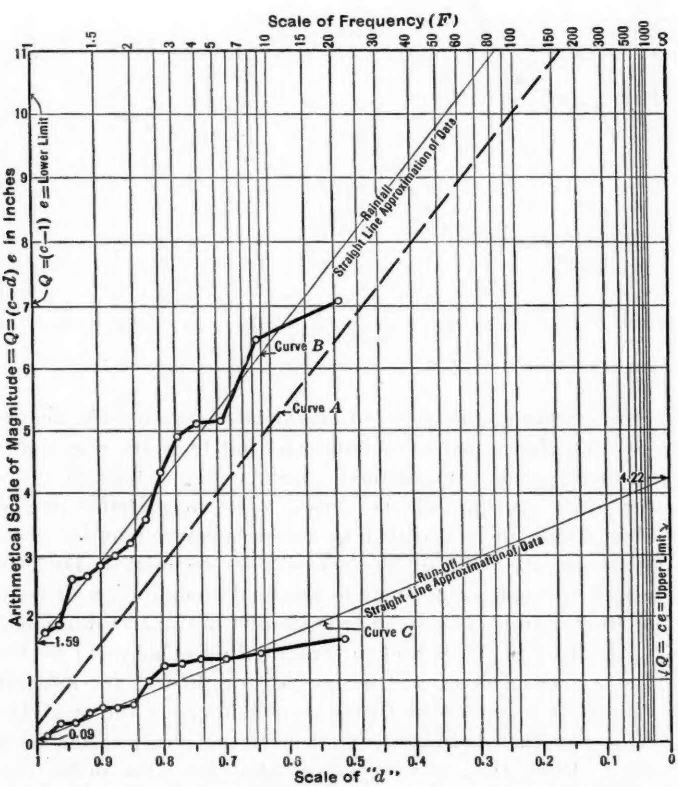

FIG. 49.—DES MOINES RIVER, KEOSAUQUA, IOWA, AFTER DRAINAGE.

Since Curve A is parallel to Curve B (Fig. 48), both curves have the same slope or the same e value in Equation (7). Since the broken straight line, A, intersects zero magnitude at the same position on the d scale as the straight line for run-off, C, both these curves have the same c value. The only difference between Equation (7) as applied to Curve A and to Curve C, is in the respective values of e for rainfall and run-off. The proportion of the remainder of rainfall (after deducting 1.34 in.) appearing as run-off, on the expectancy average for Fig. 48, is then $\frac{3.48}{8.70} = 40$ per cent. These values are given in Column (1) of Table 40.

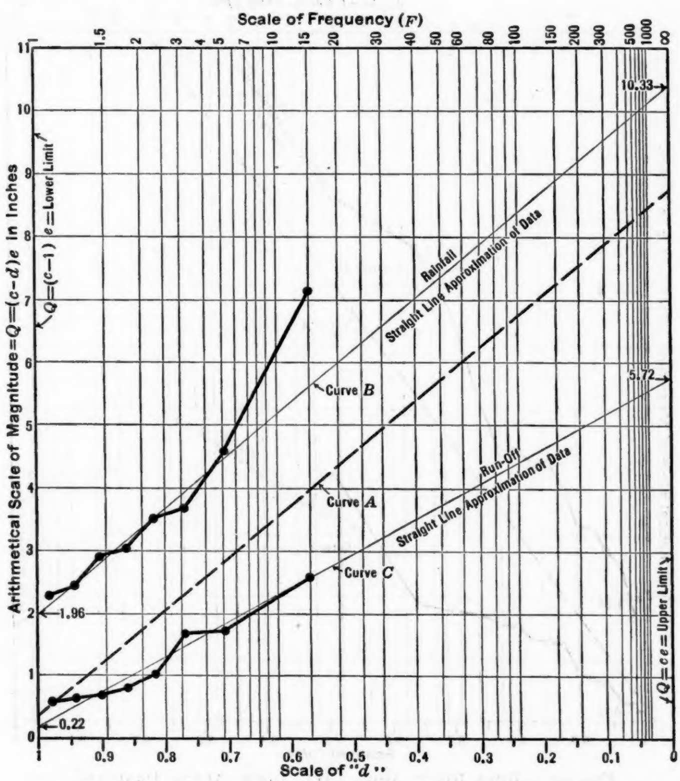

FIG. 50.—IOWA RIVER, IOWA CITY, IOWA, BEFORE DRAINAGE.

A similar process determines the values for the other columns in Table 40, and from these the following relations may be stated:

Des Moines River, Keosauqua, Iowa, Area = 13 900 sq. miles:

Before drainage, the run-off = 40.0% of (rainfall — 1.34 in.)

After drainage, the run-off = 32.6% of (rainfall — 1.31 in.)

Iowa River, Iowa City, Iowa, Area = 3 140 sq. miles:

Before drainage, the run-off = 65.7% of (rainfall — 1.62 in.)

After drainage, the run-off = 47.7% of (rainfall — 1.68 in.)

Assuming that the run-off proportion for the whole area before drainage truly represents the run-off proportion for the undrained area after drainage, then the run-off proportion for the area affected by drainage, on the Des Moines River, is equal to:

$$D = \frac{E - A\,B}{C} = \frac{0.326 - 0.673 \times 0.400}{0.327} = 0.174$$

For the Iowa River the same run-off proportion is equal to,

$$D = \frac{0.477 - 0.535 \times 0.657}{0.465} = 0.271$$

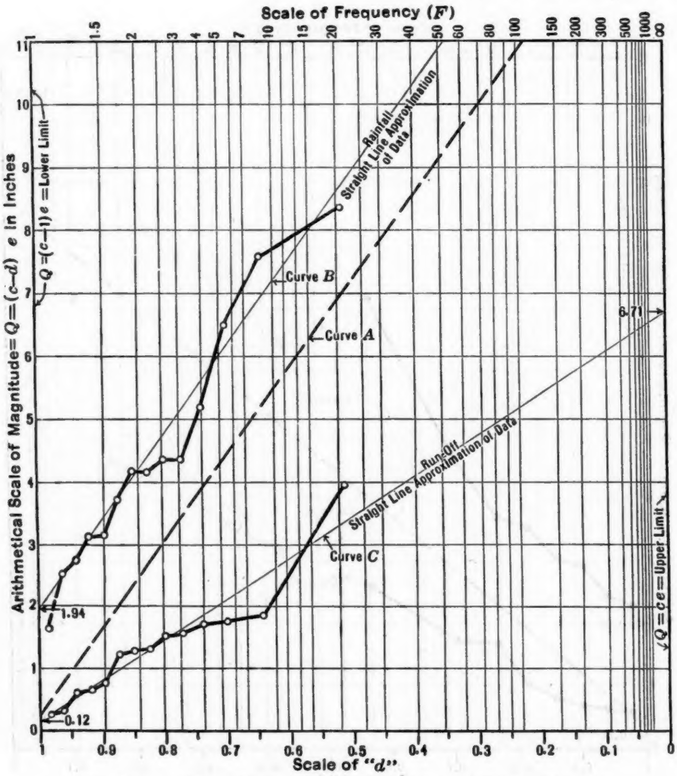

FIG. 51.—IOWA RIVER, IOWA CITY, IOWA, AFTER DRAINAGE.

After the constant deduction, the proportion of run-off from the drained areas on the Des Moines River is reduced from 0.400 to 0.174, or 43.5% of the original proportion. Likewise, the corresponding proportion for the Iowa River is reduced from 0.657 to 0.271, or 41.2% of the original proportion.

Since drainage, for agricultural purposes, is generally installed only in sections having flatter topography and slower rates of flow concentration, the assumption stated as a premise to these final computations is not exactly sound. However, the error thus introduced must be small as compared with the marked effect of drainage which the computations tend to prove.

In the very nature of the regions subjected to drainage improvements, also lies an explanation of the fact that these works show no appreciable effect on the rate of flow depletion after the conclusion of a storm. This rate would seem to be essentially a function of the more flashy portions of the catchment area, and while the writer is compelled to admire the ingenuity of the authors' attack along this line, it is difficult to conceive how drainage works on the slower areas, can be expected to show appreciable effect on this index.

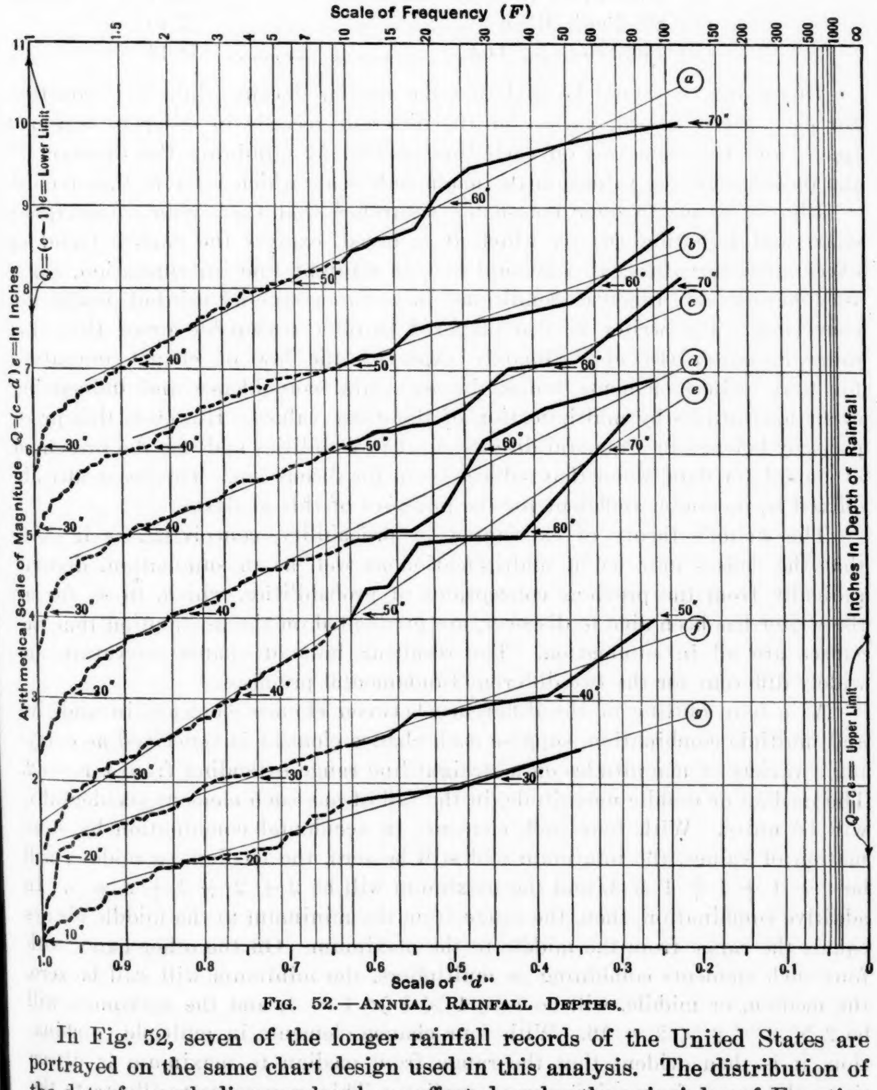

FIG. 52.—ANNUAL RAINFALL DEPTHS.

In Fig. 52, seven of the longer rainfall records of the United States are portrayed on the same chart design used in this analysis. The distribution of the data for each diagram has been effected under the principles of Equation (3), and the straight line approximating the data, in each case, has been

marked by a letter within a circle, for identification. These letters refer to the records published by the U. S. Weather Bureau, listed as follows:

Total Annual Rainfalls for:	Inches (Mean).
(*a*) Boston, Mass.	43.57
(*b*) Philadelphia, Pa.	42.70
(*c*) Marietta, Ohio	41.22
(*d*) St. Louis, Mo.	39.77
(*e*) Muscatine, Iowa	37.07
(*f*) St. Paul, Minn.	27.40
(*g*) Pembina, N. Dak.	18.14

In passing, it should be said that the zero or datum, while it is constant for each given record, varies for the different records in order to conserve space, and the numerals on each line of Fig. 52 adjoining the diagram of the record show the values on the magnitude scale which refer to that record.

Fig. 52 seems to offer reasonable assurance that the writer's expectancy chart and Equation (6) on which it is based, express the chance trend or expectancy curvature, as a general law, in straight line interpretation, for a very considerable range in conditions, in so far as annual rainfall depths are concerned. The writer is also prepared to offer reasonable proof that this same Equation (6) approximately expresses the law of chance curvature for four chance elements falling in accidental coincidence and influencing event magnitudes by multiplication of the cause values. However, this proof properly belongs in a general discussion of probabilities and may be presented at some later date, when that subject is up for discussion. The proof already offered is, no doubt, sufficient for the purposes of this analysis.

The writer's theory of expectancy or probability, conceiving, as it does, that the causes may be in multiplication as well as in summation, departs radically from the previous conceptions of probabilities, which, in so far as the writer has been able to discover, are predicated on the assumption that the causes are all in summation. The resulting laws of chance curvature are widely different for the two different fundamental premises.

As a bare outline of the difference between chance elements in additive and multiple combination, suppose each chance element is conceived as carrying a variety of magnitudes on a straight-line range extending from zero to 2. The median or middle magnitude, in the fall of one such element accidentally, will be unity. With four such elements in accidental combination by summation of values, the minimum will still be zero, the median, or middle, will be $1 + 1 + 1 + 1 = 4$, and the maximum will be $2 + 2 + 2 + 2 = 8$. In additive combination, then, the range from the minimum to the middle always equals the range from the middle to the maximum. On the other hand, with four such elements combining as multipliers, the minimum will still be zero, the median, or middle, will be $1 \times 1 \times 1 \times 1 = 1$, and the maximum will be $2 \times 2 \times 2 \times 2 = 16$. With four chance elements in multiple combination, it is then evident that the range from median to maximum is fifteen times the range from minimum to median. This may serve to illustrate the radical difference in the curvature under the two conceptions.

It is common knowledge that in the mathematical interpretation of natural phenomena, the known cause elements, especially when rates or velocities enter into the determination, are generally factors in multiplication, instead of items in summation, and it is this fact that tends to support the writer's theory.

The writer hopes that this outlook on the problem under consideration may prove of interest. The subject outlined is endless, and the effort has been to call attention merely to some of the outstanding links in the chain of evidence.

The expectancy method applies, or may be adjusted to apply, to a great number of problems in which a variety of magnitudes are encountered. By this means, for instance, the economic design of engineering works dealing with rainfall and its consequences, the best bargain in capacity for combatting or utilizing the run-off may be selected with a degree of certainty that other methods cannot hope to approach. Beyond this, the field of usefulness is perhaps still speculative, but astonishing developments in extending the scope of human knowledge may reasonably be anticipated.

WILLIAM M. HALL,* M. AM. SOC. C. E. (by letter).†—This discussion describes the formation of floods and flood crests on the Upper Ohio River, their movement, duration, and relation to the great floods and great flood crests on the Mississippi River. It invites attention to a condition of the movement of storms from west to east over the Ohio Valley which makes it appear probable, if not apparent, that in the case of the Ohio, better service will result by locating some if not all the detention reservoirs on the tributaries entering the Ohio above Cincinnati, Ohio, rather than below Louisville, Ky. It also undertakes to show that the crest wave, or water above the flood line, is the only part of the great volume flowing down the Ohio which it is advisable or necessary to catch in detention reservoirs in order to abate the great flood damage on the Ohio; that it may be caught, and, if caught on the Upper Ohio tributaries and held for about two weeks, it will probably reduce the height of floods on the Mississippi by several feet, thereby greatly increasing the factor of safety for the flood protection project proposed by Maj-Gen. Edgar Jadwin, Chief of Engineers, U. S. A., M. Am. Soc. C. E.

From the Symposium on Flood Control,‡ it appears that some additional available data on the subject showing the formation and movement of the flood crests may be useful in a further study of reservoir sites in the Ohio Valley.

Presumably engineers are aware that nearly all the damage by floods along the Ohio and the Mississippi is done by that part designated as the crest wave, which is formed mainly from tributary water and flows above the flood line or the flood stage plane established by the U. S. Weather Bureau and shown on its daily reports for the gauges at the principal cities and towns along the rivers from Pittsburgh, Pa., to New Orleans, La. To eliminate the damage caused by a flood it is only necessary to reduce or elimi-

* Parkersburg, W. Va.

† Received by the Secretary, April 7, 1928.

‡ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2451.

nate the crest wave sufficiently; or, in other words, temporarily to detain in reservoirs that volume of water which would flow above the flood stage, as designated by the Weather Bureau.

In general, the floods in the Ohio Valley are produced by storms which approach the Ohio water-shed from the southwest, travel across it from its southerly or westerly boundary to its easterly or northeasterly boundary, a distance of 600 to 800 miles. This meteorologic condition is so uniform it seems safe to assume that, of the great floods, 90% or more occurred from storms which passed from the southwesterly edge of the drainage area to the easterly or northeasterly boundary, occasionally being augmented by a storm from the west or northwest joining the one from the southwest in the area north of the Ohio. Such a condition caused the great flood of 1913. This movement of storms from west to east usually consumes a period of 2 to 4 days from the Mississippi River to the easterly boundary of the Ohio water-shed.

The assembly of a part of the available data on the actual formation and movement of the greatest floods in the Ohio Valley since 1884 appears to explain the formation and movement of flood crests. It may be useful in forming a final conclusion as to the location of reservoirs in the Ohio Valley for reducing flood crests in the Mississippi, which reservoirs will greatly relieve the Ohio of its present greatest unconquered menace and affliction without undue cost to the Mississippi project.

The writer appreciates the great accomplishment of the authors of the Symposium; also the great amount of work involved in the formulation of a complete report and estimate of cost of the flood control of the Mississippi River in its alluvial valley in so short a time for its presentation to Congress by General Jadwin. Therefore, it would be most surprising if some details of the many elements which affect the problem may not at first have been given too little value. Moreover, as has been well said, it is perfectly sound engineering to consider first the repairs to the levee system, to provide spillways or other adequate ways for the release and quick discharge of all the water which may converge from every source in the Lower Mississippi, and thereafter to treat, as an additional factor of safety, the location and building of reservoirs or other aids for the reduction of the floods at Cairo, Ill., and thereby throughout the alluvial valley. In this connection it is well to remember that the flood record of 75 years is very brief for estimating what may occur in the coming 500 to 1 000 years; therefore, it appears that the factor should be made adequate. The writer's work of 20 years on the improvement of the Ohio has involved a familiarity with the formation, movement, and character of its floods and their intimate relation with the Mississippi floods. The additional data here presented may be useful in making the best and possibly the most economical solution of the problem.

In Table 41, the movement from day to day of the crests of thirteen of the largest floods from 1884 to date is shown for nineteen gauge stations, from Pittsburgh to New Orleans. The gauge readings are taken from the records of the U. S. Weather Bureau reports, and from the Mississippi River Commission reports of "highest and lowest annual stages".

The same information is shown graphically in Fig. 53. It also shows the water surface on March 29, 1913, the day of greatest flood stage at Marietta, Ohio, and Parkersburg, W. Va., from which is more easily visualized the effect on that flood of catching the water for five days or more which passed above the flood line from March 27 to April 2 at Catlettsburg, Ky.

An inspection of Table 41 and Fig. 53 in connection with Colonel Kutz' Tables 13* and 14,† with a map (Fig. 54) makes it obvious that all or nearly all the great floods in the Mississippi River and the Lower Ohio River have their beginning in the Upper Ohio water-shed. By a comparison of the data it is observed that most of them are not only formed in the Upper Ohio River above the mouth of the Scioto, at Portsmouth, Ohio, but from that locality to Helena, Ark., above the mouth of Arkansas River, the movement down stream is very uniform from all the floods except those of 1897, 1916, and 1920, the movements of which were somewhat checked.

In the flood of 1897 the time of crest at Evansville, Ind.—7 days from Pittsburgh—was prolonged to 28 days in reaching its peak at Cairo, because the very large flood from the Tennessee was unusually slow in arrival at the Ohio. In 1920, the slow movement of 17 days to Cairo appears to be due to a slow and large discharge of several tributaries. In 1916, the 21 days difference in time of crests at Pittsburgh and Cairo was because the large discharge from the Mississippi (about 41% as large as from the Ohio) was slow in arrival at Cairo. Of the floods tabulated, those of 1916 and 1927 appear to be the only ones in which the Mississippi affected the time of the arrival of the Ohio crest at Cairo, or caused any delay in its movement down stream. The delay was nearly 2 weeks in 1916 and about 2 days in 1927. The quick arrival of the Ohio crest at Cairo in 1907 and 1904 was due to the formation of crests at Pittsburgh on or about the same dates as at Portsmouth, 338 miles farther down stream, which, in turn, was due to unusually large discharges by the tributaries from the Big Sandy to the Kanawhas.

From Table 41, for the thirteen floods the average difference in time of the flood crests at Pittsburgh and Cairo (978 miles) is 13.9 days; omitting the three slow movements in 1920, 1916, and 1897, the average time of passage of crests from Pittsburgh to Cairo is 11.4 days, a minimum of 8 days and a maximum of 16 days. This gives an average velocity of 86 miles per day for the crest movement. From Cincinnati to Cairo, 510 miles, the average difference of time for the thirteen floods is 10 days, or 51 miles per day. From Parkersburg, as far down the Mississippi River as Helena, the movement is fairly uniform, the average time for the 1 285 miles being 23 days, or 47.8 miles per day, the fastest movement being 16 days for the 1907 and 1927 floods, and the slowest, 38 days in 1897. From Helena to New Orleans, 657 miles, the average difference in time of passage of the flood crests given as 21 days is obtained without including the 1927 flood, when an early crest was produced by crevasses and an artificial by-pass which effectually stopped the rise at New Orleans 2 days before the head rise reached Helena.

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2501.
† *Loc. cit.*, p. 2502.

TABLE 41.—TIME OF PASSAGE OF CRESTS OF GREATEST FLOODS IN OHIO RIVER FROM SOURCE TO MOUTH, AND ON MISSISSIPPI RIVER FROM CAIRO, ILL., TO NEW ORLEANS, LA.

Name.	Distance, from Pittsburgh, in miles.	Flood stage, in feet.	DATE OF CREST OF FLOOD. 1927.		1922.		1920.		1917.		1916.		1913.		1912.	
			Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.
Pittsburgh, Pa.	0	25	4/7	16.8	3/16	18.3	3/14	24.5	3/13	23.0	1/13	17.1	3/28	30.4	3/31	17.5
Beaver, Pa.	29	30	4/8	15.8	3/16	23.7	3/14	36.3	3/14	32.2	1/14	25.5	3/28	46.3	3/31	26.7
Wheeling, W. Va.	95	43	4/8	17.3	3/17	23.2	3/15	38.5	....	....	1/14	29.6	3/28	51.1	4/1	27.0
Marietta, Ohio	171	33	....	....	3/16	27.0	3/15	34.4	....	....	1/15	29.2	....	....	....	....
Parkersburg, W. Va.	184	36	4/10	18.3	3/16	30.0	3/16	34.9	3/15	38.4	1/15	30.8	3/29	58.9	4/1	29.2
Point Pleasant, Ohio	265	40	4/11	28.7	3/17	39.3	3/17	40.4	3/16	47.1	1/15	37.5	3/30	62.8	4/5	40.8
Catlettsburg, Ky.	319	50	4/11	37.5	3/17	49.0	3/21	53.5	....	....	1/15	45.7	3/31	67.7	4/4	49.2
Portsmouth, Ohio	355	50	4/12	37.7	3/18	49.9	3/21	52.9	3/17	54.4	1/15	49.3	3/31	67.9	4/5	49.8
Cincinnati, Ohio	468	52	4/13	39.3	3/18	52.2	3/22	54.6	3/17	56.1	1/15	53.1	4/1	69.8	4/6	51.7
Louisville, Ky.	602	8	4/14	16.9	3/19	30.2	3/23	31.5	3/19	30.5	1/15	31.2	4/2	44.9	4/7	28.5
Evansville, Ind.	792	35	4/16	36.3	3/21	42.9	3/25	42.8	3/22	42.9	1/18	43.6	4/5	48.4	4/9	42.5
Paducah, Ky.	934	43	4/18	47.2	3/24	48.8	3/28	45.3	3/25	47.1	1/19	45.7	4/7	54.3	4/8	49.9
Cairo, Ill.	978	45	4/20	56.4	3/26	53.6	3/31	51.4	4/4	50.1	2/4	53.4	4/7	54.8	4/6	54.0
Memphis, Tenn.	1 205	35	4/22	45.9	4/1	42.5	4/5	40.3	4/10	40.4	2/9	43.4	4/9	46.5	4/6	45.2
Helena, Ark.	1 285	44	4/26	56.7	4/3	53.1	4/8	50.1	4/12	49.9	2/11	53.4	4/22	55.2	4/22	54.3
Vicksburg, Miss.	1 580	45	5/4	58.7	4/27	54.8	4/27	50.9	4/23	50.0	2/15	53.8	4/27	52.3	4/12	51.6
Natchez, Miss.	1 683	46	....	....	4/26	55.3	4/28	51.5	4/25	49.9	2/15	53.5	4/27	52.6	4/14	51.4
Baton Rouge, La.	1 819	35	....	....	5/16	45.7	5/22	41.5	4/29	37.8	3/1	42.6	5/9	41.3	5/11	43.8
New Orleans, La.	1 942	17	4/24	21.0	4/25	21.3	5/18	19.2	5/2	16.9	3/3	20.1	5/5	20.3	5/11	21.1
CREST MOVEMENT, NUMBER OF DAYS.																
Pittsburgh, Pa., to Cairo, Ill.	....	....	13		10		17		12		21		10		10	
Cincinnati, Ohio, to Cairo, Ill.	....	....	7		8		9		8		19		7		3	
Parkersburg, W. Va., to Helena, Ark.	....	....	16		18		23		28		26		22		21	
Helena, Ark., to New Orleans, La.	....	....	— 2		22		40		20		21		13		19	

TABLE 41.—(*Continued.*)

Name.	Distance, from Pittsburgh, in miles.	Flood stage, in feet.	DATE OF CREST OF FLOOD.											
			1907.		1904.		1903.		1898.		1897.		1884.	
			Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.	Date.	Stage.
Pittsburgh, Pa.	0	25	1/20	23.2	3/28	15.2	3/1	28.9	3/24	28.5	2/24	28.9	2/6	31.9
Beaver, Pa.	29	30	1/20	33.7	3/28	23.6	3/1	38.1	3/24	42.3	2/24	37.7	....	....
Wheeling, W. Va.	95	43	1/20	36.1	3/28	24.2	3/2	40.2	3/24	43.9	2/25	37.0	2/8	46.5
Marietta, Ohio	171	33	....	....	....	....	....	....	....	....	2/25	36.0	2/9	52.0
Parkersburg, W. Va.	184	36	1/21	39.9	3/29	26.6	3/3	39.4	3/26	47.8	2/25	37.5	2/9	55.
Point Pleasant, Ohio	265	40	1/19	52.0	3/29	30.3	3/4	45.0	3/27	51.2	2/25	52.3	....	....
Catlettsburg, Ky.	319	50	1/20	59.9	3/28	36.4	3/4	49.7	3/28	56.0	2/25	58.6	....	....
Portsmouth, Ohio	355	50	1/20	60.9	3/29	39.8	3/4	51.3	3/28	57.3	2/25	59.0	....	....
Cincinnati, Ohio	468	52	1/21	65.1	3/27	45.7	3/5	53.2	3/29	61.4	2/26	61.1	2/14	71.1
Louisville, Ky.	602	28	1/22	41.4	3/28	22.9	3/9	28.5	3/30	36.3	2/28	35.4	2/16	46.6
Evansville, Ind.	792	35	1/24	46.2	4/3	39.8	3/11	42.4	4/2	44.8	3/2	43.6	2/18	48.8
Paducah, Ky.	934	43	1/28	45.7	4/4	44.7	3/15	47.6	4/6	47.3	3/24	50.9	2/23	54.2
Cairo, Ill.	978	45	1/28	50.4	4/5	49.1	3/15	50.6	4/6	49.8	3/25	51.6	2/22	51.8
Memphis, Tenn.	1 205	35	2/8	40.3	4/11	39.2	3/20	40.1	4/10	37.3	3/20	37.6	3/1	34.2
Helena, Ark.	1 285	44	2/6	50.4	4/15	47.6	3/26	51.0	4/17	49.1	4/4	51.7	3/6	47.0
Vicksburg, Miss.	1 580	45	2/12	49.6	4/25	46.8	3/28	51.8	4/23	49.4	4/16	52.5	3/25	49.0
Natchez, Miss.	1 683	46	2/13	48.9	4/25	45.5	3/28	50.3	4/26	47.4	5/2	49.8	3/26	47.4
Baton Rouge, La.	1 819	35	2/17	37.4	4/29	33.7	4/7	40.1	4/29	34.6	5/13	40.6	3/24	36.2
New Orleans, La.	1 942	17	2/16	18.6	5/1	16.1	4/6	19.4	5/1	17.0	5/13	19.5	3/18	15.6

CREST MOVEMENT, NUMBER OF DAYS.

Name.	Distance, from Pittsburgh, in miles.	Flood stage, in feet.	1907.	1904.	1903.	1898.	1897.	1884.
Pittsburgh, Pa., to Cairo, Ill.	....	....	8	8	14	13	28	16
Cincinnati, Ohio, to Cairo, Ill.	....	....	7	9	10	7	27	8
Parkersburg, W. Va., to Helena, Ark.	....	....	16	17	23	22	38	25
Helena, Ark., to New Orleans, La.	....	....	10	16	11	16	39	12

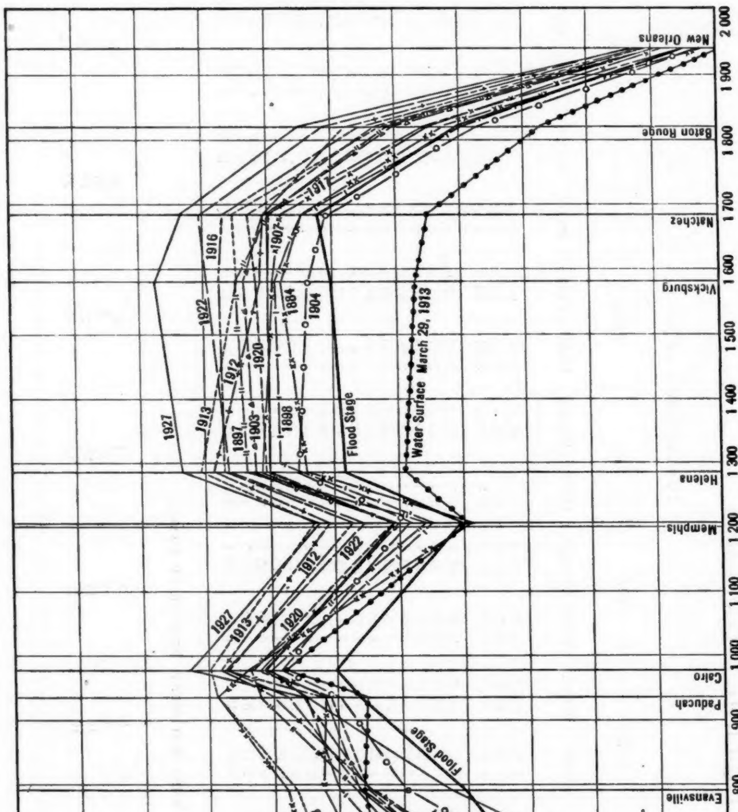

FIG. 53.—FLOOD STAGES OF THE OHIO AND MISSISSIPPI RIVERS.

From these data, personal observation, and study of flood movement, the writer believes that in great rivers like the Ohio, between banks from ¼ mile wide at Parkersburg to 1 mile wide near Cairo, and the Mississippi, from ½ to 1⅓ miles wide between Cairo and the mouth, the great mass of water, flowing 20 ft. or more above the bed of the river and 300 ft. or more from the shores, is so little affected by the friction of bed and banks that the velocity is nearly uniform throughout any section, except where it is more or less disturbed by the inflow from large tributaries and by going over the 26-ft. fall at Louisville.

As may be observed from Fig. 54, between the Scioto, at Portsmouth (above which the governing crest peak is usually formed) and Cairo, 610 miles, there are only seven tributaries large enough to cause much disturbance in the crest wave; or, including the Falls, only one dislocation of relative position of the particles of water in the crest wave for each 75 miles of river. It is believed, therefore, that in any great flood crest passing Portsmouth, if the particles of water in any section of the crest, 100, 200, or 300 miles long, could be located and their relative position determined, when passing Cairo, it would be found that in the corresponding section nearly all the water in crest near Portsmouth would be found near Cairo, of course, augmented by the intermingling of the increments of inflow from tributaries.

In a discussion of this subject* the late James A. Seddon, M. Am. Soc. C. E., designated such condition as "perfect flow". The crest wave of approximately so-called perfect flow is probably about 1 000 ft. wide in the section of river from Catlettsburg, at the mouth of the Big Sandy, to Cincinnati, and about 2 200 ft. wide below Paducah, at the mouth of the Tennessee. Due to this increase of width and also to an increase of several miles in width of back-water into side streams and overflowed bottom-land from Paducah to Cairo and down the Mississippi, the effect of catching any definite part of the crest-wave water in the upper section of the river cannot be estimated with accuracy at Cairo.

Although topographical maps of the area up to the high-water plane are not available, it is known that the flood-plane broadens out to a width of several miles over which this crest wave must spread at stages above 45 to 50 ft.; and at its entrance to the Mississippi, it must flow in between levees about 1¼ miles apart. With such approximate data, the water surface area for that 100 miles of river, or the acre-feet of water per foot of depth, is estimated to be approximately 136 000. It appears, therefore, that should the daily discharge of the crest wave from above Cincinnati be reduced by 300 000 to 400 000 acre-ft. (or 150 000 to 200 000 cu. ft. per sec.) for a period of from 3 to 10 days, while the peak of the crest is passing Cairo and entering the Mississippi, the crest height would certainly be reduced by 2½ to 4 or 5 ft., the exact amount depending upon local conditions, such as the stage of the crest peak, the discharge of the Mississippi River above the Ohio, the impounding effect of the levees, etc. Therefore, observations of flow and comparisons of discharge at respective localities appear to be the best guide, until more

* Annual Report, 1892, U. S. Chf. of Engrs., p. 2907.

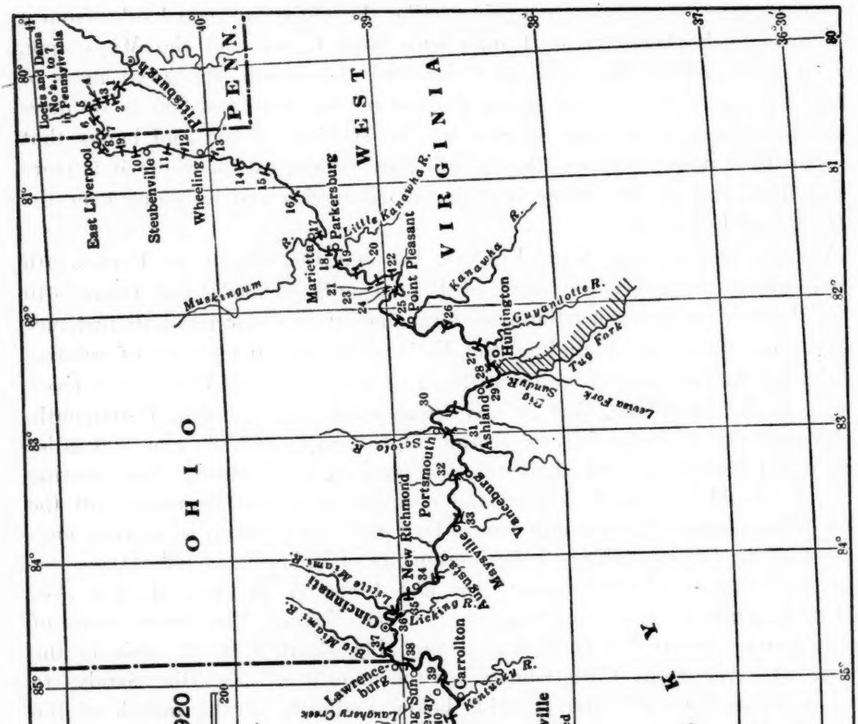
MAP OF
OHIO RIVER
REVISED TO SEPTEMBER 30, 1920
SCALE IN MILES
0 50 100 200
OHIO
INDIANA
ILLINOIS
KENTUCKY
WEST VIRGINIA
PENN.
Locks and Dams No. 2, 1 to 7 in Pennsylvania
Pittsburgh
East Liverpool
Steubenville
Wheeling
Marietta
Parkersburg
Point Pleasant
Huntington
Ashland
Portsmouth
Vanceburg
New Richmond
Maysville
Augusta
Cincinnati
Lawrenceburg
Rising Sun
Vevay
Carrollton
Madison
Louisville
42 Eliminated
Leavenworth
Owensboro
Evansville
Henderson
Paducah
Mound City
Cairo
Muskingum
Little Kanawha R.
Kanawha R.
Guyandotte R.
Tug Fork
Big Sandy R.
Levisa Fork
Scioto R.
Little Miami R.
Big Miami R.
Licking R.
Laughery Creek
Kentucky R.
Falling Run
Salt R.
Big Blue R.
Green R.
Wabash River
Tradewater R.
Saline R.
Cumberland R.
Tennessee R.
Cache R.
Mississippi River

FIG. 54.

exact information is obtained as to areas flooded at high stages. The reader may more readily visualize the effect simply by inspecting Fig. 53 and imagining all the water withheld from Beaver, Pa., to Louisville enclosed by the flood-stage line and the various crest lines.

A study of the greatest floods which passed Cincinnati, 468 miles below Pittsburgh, from 1870 to 1921 (Fig. 55), shows that there were thirty-eight up to or above the flood line (52 ft.) and that their duration above that line was from 1 day to 18 days; that six of them were from 1 day to 10 days above the 62-ft. stage (10 ft. above flood stage), the total being only 30 days in 51 years. At Cincinnati, the fluctuation between high and low water is the greatest on the Ohio, and the duration of floods is probably as great or greater than at any other locality above Paducah at the mouth of the Tennessee.

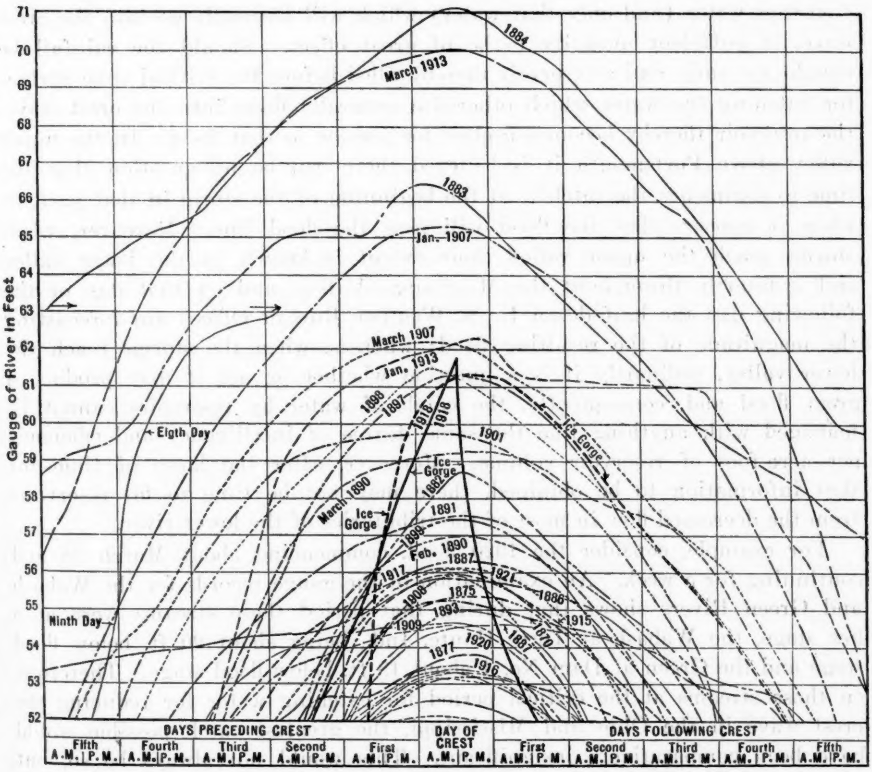

FIG. 55.—FORMATION AND MOVEMENT OF FLOOD CRESTS IN THE OHIO AND MISSISSIPPI RIVERS, SHOWN BY HYDROGRAPHS AT CINCINNATI, OHIO.

Thus, the crest waves, which do practically all the damage along the Ohio, are generally of short duration. As compared with the great volume of water which goes down the river, it is evident that only a small fraction of 1% needs to be caught in order to eliminate the damage; and with one exception in 58 years it has passed in the four months, January to April,

inclusive. By a similar study, the passage of the largest floods by Cairo is found to range with one or two exceptions from 12 to 31 days. Therefore, it is probable that the water which passes Cairo before the stage has reached the flood line, that is, prior to 5 to 10 days before the passage of the peak of the crest, ordinarily does no damage. It certainly does no damage unless it happens to meet a discharge from one or more of the largest tributaries down stream sufficient to produce a flood stage in the lower river; or, unless bottled up by insufficient outlets, a condition which will be relieved by General Jadwin's plan.

Due to this formation and movement of flood crests from the upper river and the condition of ordinarily having rain storms in the Lower Ohio Valley several days prior to those in the upper valley, reservoirs built in the lower valley will probably be found more difficult to fill at just the right time to stop that water (and only that water) which will naturally go into the crest wave, in sufficient quantity to be of great effect. Should the rainfall be caught too early and a reservoir thereby filled before the critical time arrives for catching the water which otherwise naturally flows into the crest wave, the reservoir thereby becomes useless for service to that flood. In the upper valley above Portsmouth it is believed there can be no question that the time to commence the catch is at the beginning of the storm in that part, or when it appears that the flood will pass the flood line. Moreover, when storms reach the upper valley their extent is known in the lower valley and ordinarily throughout the Mississippi Valley, and on that day or the following day the half dozen U. S. Weather Bureau Offices are forecasting the magnitude of the resulting flood; whereas, when the storms reach the lower valley, ordinarily it is a question whether or not it may produce a great flood and, consequently, the catch of water by reservoirs cannot be managed with anything like the same degree of intelligence and efficiency per acre-foot of reservoir volume. Moreover, after the lapse of time for that information to be obtained, there may not be time to fill reservoirs from the decreased flow in most of the tributaries of the lower river.

For example, consider the 1920 flood, commencing about March 28 and continuing for a week. An examination of the gauge records for the Wabash and Green Rivers shows that during that period these streams were at a low stage, the Wabash at Terre Haute, Ind., being about 20 ft. below flood stage and the Green at Dam No. 4 about 18 ft. below flood stage. Therefore, on those streams at the critical period for catching water for reducing the crest wave in the Ohio and Mississippi, the greatest catch possible would have been comparatively insignificant. This would not always be so, but it appears that it would frequently be so. The data on those streams for the earlier floods are not available. It is not claimed that a reservoir on the Wabash or on the Cumberland or on the Tennessee would not be desirable or effectual for some floods or be inadvisable to build, but it is believed the condition named will result in more difficulty in their efficient operation, or in less efficiency than for those in the upper river. Regularity of the movement of the crests from Pittsburgh or above Portsmouth to Cairo

largely reduces the objection to the greater distance from Cairo of detention reservoirs on the upper tributaries between the Scioto and the Beaver.

Moreover, should it be determined to operate reservoirs on the lower river solely for the benefit of the Mississippi, that is, to let the crest wave in the tributary run out without restraint regardless of its local damage, and thereafter catch the water which would naturally go into the Ohio and Mississippi River crest wave, a serious conflict of interest might result. It is for this reason that it has been concluded that the Miami River Conservancy cannot be operated for the protection of the Miami Valley and at the same time for the protection of the Mississippi River.

Data for accurately estimating the quantity of water in the crest wave are not available, therefore, only an approximate estimate is possible.

The greatest flood of record at Pittsburgh was in 1907 and the greatest on the Mississippi, in 1927, and the 1913 flood is the greatest of record from above Wheeling to near Cincinnati. At 6:00 P. M., on March 29, the 1913 flood was at its crest of 58.3 ft., 22.3 ft. above the flood stage at Parkersburg and about 25 ft. above flood stage at Marietta; on the same day it was slightly below the flood stage at Coraopolis, Pa., 9 miles below Pittsburgh; 7 ft. above flood stage at Wheeling; 20 ft. above at Point Pleasant; 16 ft. above at Portsmouth; 14 ft. above at Cincinnati; 13 ft. above at Louisville; 8 ft. above at Evansville; and at just flood stage at Paducah, possibly the greatest quantity of water ever in that part of the river, 925 miles. In the part between Coraopolis and Louisville, 593 miles, a distance of flow of about 5 days, it is believed, if the water in the crest wave (above the flood line) could have been caught on the tributaries above Portsmouth and held for 15 or 20 days, the time required to flow to Helena, and then gradually released, the damage from that flood would have been eliminated in the Ohio and materially reduced in the Mississippi, probably by 2½ to 4 ft. in stage at Cairo. As the average depth of the crest wave above the flood stage on that day was about 12½ ft., for the 593 miles, the volume of water in the crest wave or above flood line was about 2 000 000 acre-ft. On that date the discharge of the Ohio under the Baltimore and Ohio Railroad Bridge, at Parkersburg, was probably about 700 000 sec-ft., of which probably about 300 000 sec-ft. was in the crest wave above the flood line. This estimate is based on an inspection of the river, by traversing it in a boat, and from memory of discharge observations and velocities at various low-water and high-water stages up to 45 ft. at Dam 18, 5 miles up stream.

An independent measurement* gives the discharge at Parkersburg for 7 days, March 26 to 31, as 323 654 400 000 cu. ft. (7 450 000 acre-ft.) which harmonizes with the writer's estimated rate of discharge. Applying those rates for the 5 days the river was above the flood stage at Parkersburg and Marietta, it is found that about 1 490 000 acre-ft. passed that locality flowing above the flood-plane and 5 960 000 acre-ft. flowing below the flood-plane; or, if 1 500 000 acre-ft. of that water had been caught at the proper time from the Little Kanawha, Muskingum, Beaver, or other tributaries,

* *Bulletin Z*, U. S. Weather Bureau, 1913.

and held for 10 days or more before releasing, there would have been no flood damage in that part of the Ohio; and, thereby, a corresponding reduction would have occurred in the crest height to the mouth of the Ohio and in the Mississippi until the crest reached the water impounded by the action of the levees. The discharge of the same flood wave at Cincinnati (Mile 470) above the flood-plane, was a little more than double the corresponding discharge at Parkersburg, that is, somewhat in excess of 3 000 000 acre-ft. It was 11 days in passing, and its peak was 17.9 ft. above the flood line. Had that 3 000 000 acre-ft. been caught and held for 15 to 20 days, it would at least have eliminated most of the damage above Paducah and beyond reasonable doubt would have reduced the crest by several feet from Cairo to Helena, Ark., or farther down, if it had not become impounded by levees.

Should a great flood in the Mississippi be mainly produced from other tributaries as occurred in the exceptional case of the 1927 flood, a detention reservoir system in the Upper Ohio Valley could be made effective in reducing stages in the Mississippi by closing the detention dams when that condition was observed from the rainfall or from gauge readings of its other tributaries. It would be entirely practicable to fill the reservoirs, provided the flow in the Ohio was not reduced to a stage below a 10-ft. navigable stage, a very improbable condition. For example, when the flood of 1927 was forming, it was known 10 days before the crest peak passed Cairo that a great flood was approaching, and 5 days before, it was remarked in the Louisville Engineer Office that the previous record at Cairo would probably be broken.

As the Upper Ohio Valley along the banks of the Ohio River is much more thickly populated than either the part below Louisville, or the Mississippi Valley, and as above Parkersburg the greater part of its banks are densely occupied by industrial plants, the money loss by floods is probably greater per mile of river than from Louisville to New Orleans. The money damage to Dayton and the Miami Valley by the same flood has been said to be from \$30 000 000 to almost \$100 000 000. By the same measure for estimating, the loss in the Ohio Valley above Cincinnati by the same flood must have been much more.

It should be realized that on the way down these two great rivers, 1 942 miles from Pittsburgh to New Orleans, it is the same crest water that passes Pittsburgh, Marietta, Parkersburg, Cincinnati, Louisville, and Cairo, doing such vast damage in its passage; and it is a large factor, usually the principal factor, in the formation of the crest wave at Cairo (its contribution in the thirteen floods tabulated being from 51 to 87% of the water passing Cairo at crest stages), which proceeds on down the Mississippi to New Orleans, producing many more millions of damage and untold human suffering. Now seems to be the time for the National Government to join with the Ohio Valley States in eliminating this great menace to property, life, and happiness, by managing it as a part of the larger Mississippi River project.

To those familiar with the construction and operation of a series of locks and movable dams for navigation, such as that on the Ohio, the problem of a system of detention reservoirs, appears little or no more difficult; certainly not so difficult as the proposed Ohio System appeared

twenty-five years ago. No one can read the Weather Bureau meteorologic report of the rainfall which produced the 1913 flood without concluding that a greater flood is a possibility if not a probability. That condition, combined with the fact that population and property values are continually rapidly increasing, makes it appear that the construction of detention reservoirs is only a matter of time. If so, it is possible that land values required for the reservoir sites may increase more rapidly than the interest on the money required for construction.

An estimate of cost of catching 2 000 000 or 3 000 000 acre-ft. of water from any one of the major floods noted is an engineering problem of no greater difficulty than many others which have been successfully solved by many engineers employed by the Government and by many other members of the Society. Of course, such provision for future floods is more difficult, but from a study of the records of all recent great floods over the entire valley, it appears that a system of reservoirs may be devised that will completely or materially reduce the destructive part of every crest wave, and reduce its height at Cairo by several feet; provided the reservoirs are operated so as to be filled from the water which would otherwise go into the crest wave.

It is a customary practice to forecast for the dams under construction on the Ohio, the probable rise from an up-river crest 100 to 500 or 600 miles distant. In the accuracy of such forecast is frequently involved the flooding of coffer-dams, the suspension of construction, and the lay-off from work of hundreds of workmen. From the charts used for such forecast prepared from the record of actual floods the following data are abstracted: Storms producing a flood crest at Catlettsburg, Mile 319, of 31.4 ft., on September 9, 1926, produced rises, as follows:

At Dam 46, Mile 757, about 4½ days later, 8 ft. to a stage of 21 ft.
At Dam 52, Mile 939, about 7 days later, 7 ft. to a stage of 22.5 ft.
At Dam 53, Mile 963, about 7½ days later, 7 ft. to a stage of 29 ft.

Data from this and similar floods were plotted for use as guides in forecasting because the conditions of discharge from tributaries during the periods of their flow appeared to be near the normal. It is realized that the rises indicated are not entirely due to the rise at Catlettsburg, but evidently they are largely due to it. Hence, if all the water which produced the rises at Catlettsburg had been caught and held for a period of 5 to 8 days and then gradually released, no rise therefrom could have occurred at Dam 53, 16 miles above Cairo, where gauge readings fluctuate nearly alike by changes in discharge from above Cairo on either river.

There are no data for determining with exactness the reduction in rise at Cairo from such a catch above Catlettsburg, because no two floods on the Ohio are exactly alike, and because of tributaries the discharge from each of which is a factor in the problem. Notwithstanding, it is customary practice to make such forecasts for periods of several days and distances of several hundred miles and to name within a few hours the arrival of the

peak of the crest and its gauge reading within a few tenths of a foot. On the accuracy of such forecasts is based action of much moment.

A further reason for locating a part of the proposed reservoirs on the upper instead of on the lower tributaries of the Ohio is that they may also be made to serve as storage dams for filling the pools of movable dams immediately after raising them. As is well known to navigators on the Ohio, for several days after raising the movable dams there is usually a deficiency of depth of water in the upper part of pools for navigation. The defect has not yet been corrected. When all the locks and dams are finished in 1929, traffic may demand a remedy. It may then be found that to store water on some of the tributaries is the only remedy for this navigation defect. As such shortage of water only occurs after the flood season, flood-control dams and navigation-control dams would readily serve the dual purpose.

As to finding possible locations for dams and reservoirs, the tributaries above the Scioto are hundreds of miles long, and in a hilly and mountainous country the question of engineering may be found to be largely that of comparative costs between the best locations. For example, take the possibility of a choice being made of the Little Kanawha River Valley, the tributary entering at Parkersburg (Mile 185). Between points about 32 and about 102 miles above Parkersburg, there are no railways, no paved highways, a sparse population, no factories of great value, only one bridge, and with a difference in elevation of water level for the 70 miles of about 100 ft. With one or two dams in that valley it is probable that 20 or 25% of the desired storage may be provided, or a storage sufficient for catching the entire discharge for a period of 6 to 10 days. As the writer has traversed the valley he feels safe in stating it is almost beyond question that it will be possible to found such dams on rock ledge at or near the river-bed level.

In conclusion, the reason for building storage reservoirs in the upper valley of the Ohio in preference to the lower valley, thus increasing the factor of safety in the Mississippi River flood-control project, may be briefly stated as follows:

(*a*) The catch of the water in the Upper Ohio tributaries can probably be made with more precision as to efficiency in reducing the crest of Mississippi River floods than one in the lower tributaries.

(*b*) It will add 600 miles or more of river flood protection in the most thickly populated part of the entire Ohio water-shed.

(*c*) The storage dams for flood protection may be used, without detriment thereto, during the low-water season for navigation water supply, thereby improving navigation depths in the pools of the movable dams more quickly than can be done otherwise.

(*d*) Surveys in the Upper Ohio Valley may disclose as favorable and economical sites per acre-foot of storage as in the lower valley.

(*e*) As there is greater population and greater damage in the upper river, there is correspondingly greater wealth of property to be damaged, and, therefore, local co-operation may be more readily obtained and organized. It

probably only needs the co-operation of the Government organizing force with the Ohio River States to induce them to join in planning for the work of construction.

(*f*) Reservoirs in the upper valley will be of as much, or nearly as much, protection to the Lower Ohio River communities along the river as if they were on the tributaries in the lower valley; and their operation should be made as much as 90% efficient in catch of the actual discharge of flood crest water from each storm.

(*g*) Finally, the reduction of 2, 3, or 4 ft. in flood crests at Cairo and thence on down the Mississippi to New Orleans, by detention reservoirs in the Upper Ohio Valley, may be found to be the most positive, the most economical, and the most reliable way of producing such result for the 1 900 miles of river within those limits of great floods. For the welfare of the millions of people dwelling or employed along the Ohio and Mississippi Rivers, it is a question of as great importance as any which ever arose not involving their liberty. Now is the opportune time for its consideration by all those most concerned.

C. E. Ramser,* M. Am. Soc. C. E. (by letter).†—The subject of the excellent paper by Professors Woodward and Nagler‡ has long been the cause of a great difference of opinion among drainage engineers, based largely on personal observation, often local in character. The results presented by the authors are especially valuable since they are based on actual measurements of run-off made both before and after extensive drainage operations on the water-shed.

The writer's remarks will be confined solely to the rate of run-off or peak run-off. It is believed that the difference of opinion mentioned has been due chiefly to the fact that the rate of run-off is increased in some instances and reduced in others, depending on the nature of the drainage, governing conditions of the water-shed, such as soil and covering, and intensities and amounts of rainfall considered. As a general rule open-ditch drainage is regarded as increasing the rate of run-off, and tile drainage as decreasing it. This statement being true, it is readily apparent that combinations of the two types of drainage tend to complicate the problem greatly, since, with a certain combination of the two types, drainage would produce no effect on the rate of run-off.

In general, it may be said that the effect of both open-ditch and tile drainage on the rate of run-off is primarily governed by changes in the time of concentration for the water-shed and storage conditions. The maximum rate of run-off for a given rainfall intensity occurs when water from every part of the water-shed reaches the channel at the lower end. The rain that produces this maximum rate must continue as long as the time required for water to flow from the most remote point to the lower end of the water-shed, and the greater the intensity of this rain the greater will be the rate of run-off. It is a well-established fact that, in a general

* Drainage Engr., Bureau of Public Roads, U. S. Dept. of Agriculture, Cape Girardeau, Mo.

† Received by the Secretary, April 16, 1928.

‡ *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 165.

way, the intensity of rainfall varies inversely as the duration. From the foregoing it follows that the reduction in the time of concentration of a water-shed increases the rate of run-off. Open ditches which increase the velocity of the water reduce the time of concentration for the water-shed. Tile or sub-drainage tends to increase the time of concentration since a longer period is generally required for the water to move through the soil and sub-drainage channels than over the ground surface and natural channels. Where open ditches induce sub-drainage for long distances through pasty or sandy soils, the effect of sub-drainage would be to counteract the effect of open-ditch drainage on the rate of run-off.

The storage of water on a water-shed tends to reduce the rate of run-off. Open ditches which eliminate storage in swamps and old lake beds, or reduce the height and duration of overflows on adjoining lands, have the effect of increasing the rate of run-off for rains such as are ordinarily provided for in drainage improvements. It is conceivable that a rain could occur, with a duration greatly in excess of the time of concentration of the water-shed, that would produce practically the same maximum rate of run-off before and after drainage for certain water-sheds. In tile or sub-drainage the storage of water in the soil tends to reduce the rate of run-off to the point of saturation of the soil.

It appears to the writer that while the conclusions reached by Professors Woodward and Nagler are sound for water-sheds on which both tile and open-ditch drainage have been extensively practiced, it is not believed that the results are generally applicable to all water-sheds in the Mississippi Valley, such as the St. Francis and Little Rivers on which considerable open-ditch drainage, but comparatively little tile drainage, has been done.

T. Kennard Thomson,* M. Am. Soc. C. E. (by letter).†—It has been estimated that the flood of 1927 caused a loss of $1 000 000 000, of more than 200 lives outright, and that more than 700 000 people lost their homes. In addition, the number of people whose health was shattered and whose lives were shortened from exposure and worry, must be appalling.

While the 1927 flood was the highest, those of 1912 to 1916 and many other years were also very high. Even the so-called ordinary floods, of frequent occurrence, cause enormous direct and indirect losses. Surely, therefore, a radical remedy is justifiable.

Levees which raise the water level 50 ft. above the surrounding country are a standing menace to life and property, especially as they are so easily destroyed.

The writer has read with great interest and some sorrow the various papers of the Flood Symposium.‡ None of the very able engineers has taken the "bull by the horns" and submitted a real remedy.

The proper solution is the application of the old homely saying that, "if one horse cannot pull a load get two or more". The specific plan devised by

* Cons. Engr., New York, N. Y.

† Received by the Secretary, June 13, 1928.

‡ *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2451.

the writer is shown in Fig. 56. He is satisfied that the Mississippi River problem can be economically solved by creating four new rivers, with non-corrodible banks and bottoms, one in the approximate location of the present river, and the other three as Rivers *A*, *B*, and *C*. For the sake of clearness only a few contours have been shown and the thousands of smaller rivers and streams have been omitted.

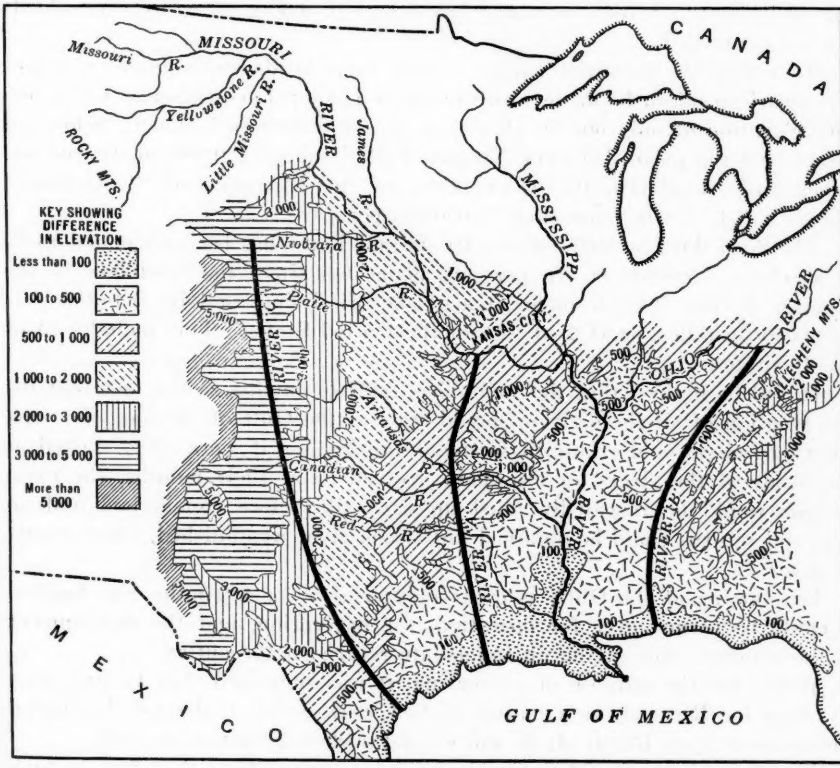

FIG. 56.—PLAN OF THE MISSISSIPPI RIVER VALLEY.

River A.—The construction of River *A* should start at the Gulf of Mexico, about 300 miles, more or less, west of the mouth of the Mississippi River; 100 miles from its mouth, it would have an elevation of 100 ft.; 300 miles, 500 ft.; and 700 miles, 700 ft. It would tap the Missouri River near Kansas City, Mo., at a cost of less than $200 000 000 per year for 20 years; but long before reaching the Missouri the profits from the construction should be realized, thus ensuring the construction of Rivers *B* and *C* and the reconstruction of the Mississippi.

River B.—The work on River *B* should commence 100 miles, more or less, east of the mouth of the Mississippi, on the Gulf of Mexico; 100 miles from the mouth of River *B* its elevation would be 100 ft., tapping the Ohio River near Ashland, Ky., or near the Big Sandy River.

River C.—River *C* should have its mouth on the Gulf of Mexico about 500 miles west of the mouth of the Mississippi; 80 miles from its mouth its elevation would be 100 ft.; 120 miles, 500 ft.; 150 miles, 1 000 ft.; 450 miles, 2 000 ft.; 600 miles, 3 000 ft.; and 1 100 miles, 4 000 ft., where it would connect with the Niobrara River. If found desirable River *C* could eventually be extended in a northwesterly direction to the Canadian border.

Engineers should control and utilize the forces of Nature instead of allowing such forces to create ruin and havoc as is usual in all cases where floods are not controlled.

To realize the potential values of such huge heads and volumes of water a comparison might be made with the Great Lakes: Lake Superior has a surface elevation of only 600 ft. above sea level (its bottom is 150 ft. below sea level), and the mean flow over Niagara Falls is about 220 000 cu. ft. per sec. When fully developed, the water power of the Niagara and St. Lawrence Rivers will be worth more than $2 000 000 per day.

The flood flow, in 1927, of the Mississippi River has been estimated to be as much as 3 500 000 cu. ft. per sec., about ten times the flood flow of the Niagara River. The drainage area of the Mississippi and its tributaries is 1 240 050 sq. miles, or 41% of that of the United States. This includes 24 of the 48 States.

It will be noticed that the writer has tried to emphasize the fact that the construction of all these rivers should start at the Gulf of Mexico. To start anywhere else would result in financial failure, for if work were started at the heads of the rivers, or elsewhere, instead of at their mouths, the excavations would be repeatedly filled up with silt before completion, and no financial returns could be obtained until each river was finished, which would be never.

By starting at the Gulf, as every section of 50 or 100 miles was finished it could be turned over for operation, for navigation, and the development of the country, thus giving an immediate return on the outlay.

While, for the purpose of estimating, it was suggested that twenty years be taken for River *A*, as a matter of fact work could, if desired, be started simultaneously on Rivers *A*, *B*, and *C*, and pushed as fast as desired.

The four rivers would naturally be interconnected so that low or high water in one could be controlled by the others. It is unusual for high water to occur in the Missouri and the Ohio Rivers at the same time, but when it does, trouble results.

A glance at Fig. 56 will show that Rivers *A* and *C* and the Mississippi would be connected by the Platte, Arkansas, Canadian, Red Rivers, etc. If found desirable a new river might be constructed between River *B* and the Mississippi, say, 200 or 300 miles from the mouth of River *B*.

Rivers *A*, *B*, and *C* should all be narrow and deep (say, 1 000 ft. wide and, at least 25 ft. deep) and should be regulated so that navigation could be carried on 365 days per year, instead of being periodically interrupted by both high and low water, as at present, to the tremendous financial loss of the country. All the rivers should have flat slopes and beds lined with concrete so as to avoid scouring of the banks, forming new channels, etc.

Silting basins should be constructed so that the millions of dollars worth of top-soil which is now lost (1 000 000 000 cu. yd. per year) can be collected and distributed where and when needed, irrigating and reclaiming waste land, etc. Two examples that follow illustrate the quantity of this silt:

First.—When building a bridge over the Ohio River near the junction of the three States of Ohio, Kentucky, and West Virginia, the latter two being separated by the Big Sandy River, the writer found low water at 6 ft. and high water at 106 ft., with no soil on top of bed-rock. A coffer-dam about 30 by 60 ft. was landed on rock, but was not sealed, before high water compelled stopping work for the winter. In the spring it was found that the coffer-dam, 25 ft. deep, was filled with silt and there was no means of telling whether it would have been filled more than once if it had been emptied as fast as filled.

Second.—In placing some pneumatic caissons for a bridge in the Missouri River, in South Dakota, the writer did not consider it feasible to dredge a channel to float the caissons from the shore to the pier sites, as the channels would fill up almost as fast as dredged; so platforms were constructed on piles from which the caissons were suspended while being built.

As the drainage area of the Mississippi and its branches is nearly 800 000 000 acres, the increased value of the land, due to navigation, irrigation, reforestation, and general development—to say nothing of the hydro-electrical possibilities—should so enhance the value of the land that these rivers would soon be fully paid for.

Furthermore, the mere undertaking of this project would tend to open up vast territories, to the benefit of the entire Continent of North America.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE VIRGINIAN RAILWAY ELECTRIFICATION

Discussion*

BY R. D. N. SIMHAM, ASSOC. M. AM. SOC. C. E.

R. D. N. SIMHAM,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The author describes the broad engineering features of the railway electrification problem quite completely and concisely. There is little more that can be said except to make a few remarks as to the merit or usefulness of a few of the details mentioned. The paper must be of immense practical help to engineers who are engaged in designing such electrification projects, particularly the data given in Tables 3§ and 5.||

The Virginian Railway electrification project is one more illustration of the economy that would be possible by changing the motive power from steam to electricity. The power then comes from a central common source; that is, the generating unit is large and comprehensive. That is the reason why, from the points of view of economy of power, equipment and operation costs, and efficiency, electrification projects invariably prove successful and beneficial both to the company and the public. The author states¶ that the Virginian electrification project has effected considerable savings by the changed form of motive power. Such conclusive evidence is very encouraging to engineers engaged in other electrification projects and should certainly stimulate engineering activities throughout the world in establishing, with confidence, electric plants in the place of steam, oil, or other forms of power plant.

So far as India is concerned, it has great prospects for developing very cheap hydro-electric power and already several undertakings of great magnitude have been started. Hydro-electric power should be cheaper than steam turbo-electric power, at least so far as the conditions of India are concerned, where good, cheap coal is scarce and natural water power is plentiful. Another advantage of water power is that it requires only the single transformation from the energy of water into electricity; whereas in steam turbo-generators

* Discussion of the paper by George Gibbs, M. Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Town Planning Asst., Madras, India.

‡ Received by the Secretary, May 18, 1928.

§ *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 27.

|| *Loc. cit.*, p. 45.

¶ *Loc. cit.*, p. 59.

a double transformation is required. The first transformation is to convert the latent energy of the fuel into heat energy as stored in steam. The next transformation is to convert the steam energy into electrical energy.

Besides the loss involved in the double transformation one of the great disadvantages of the steam-driven station is the large amount of space required for the boiler equipment and the condenser plant. In the case of the Virginian Railway plant (see Table 3), the turbine room occupies 11 440 sq. ft., whereas the boiler-room space is 13 454 sq. ft. That is, much more space is allotted to the steam-raising plant than to the generating plant. The ratio of the turbine-room space to the boiler-room space is as 100 to 118. It is usual to find the boiler occupying as much as three times the ground space required for the generators. Again—and this is specially important—boilers cannot be over-loaded 50% like turbo-alternators, so that if advantage is to be taken of the overload capacity of the generators at any time, the necessary extra boiler capacity must be secured.

In countries like America and England there are, perhaps, several reasons for the slow development of railway electrification. As stated by H. P. Gillette, M. Am. Soc. C. E.,

"Resistance to change, which usually is stronger in an old and powerful institution like the railroads than in a young and struggling one, probably has been the most serious single cause of delay; and as might be expected, it appears to have operated more effectively in England than in America".

In India, however, where railroad projects are just beginning to be developed, there is no reason for delay. All the present railway projects (which may be counted on the fingers of one hand) are capitalized by outside enterprise, and certainly the financiers would not start experimenting and would always postpone so radical a step as electrification in the hope that some other capitalist would bear the first expense of the experiment.

There is great prospect, however, that the problem of electrification will be influenced by the growth of the great power systems in India. It is recognized that the cost of power is becoming more and more important, since civilization is requiring increasingly larger amounts of cheap power per capita. It will become still more important to utilize all possible sources of natural power, which is plentiful in India and is now going to waste. Cheap fuel is scarce and the cost of transportation is high. Therefore, this problem is becoming more important and more serious each year. The development of hydro-electric power would relieve the situation tremendously. To India, electricity is indeed the only practical source of economical power. It is inexhaustible, and should displace all the old forms of motive power.

The considerations that should generally be the basis of any power plant design are well illustrated in Fig. 4.* The somewhat sharp peaks and sudden falls in the curve indicate that it must be very difficult to maintain economical operation of the station. Quite contrary to what might be usually expected in a railway load curve, the paper shows that, on two occasions within a period of twelve hours, the entire station load was taken off; but the sudden falls are not necessarily so serious in the economical running of the plant. The

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 13.

sharp peak loads, especially those of very short durations, are, however, very disadvantageous. For the sake of supplying power just for the abrupt peak loads, all the generators must operate and thus there is a great deal of idle power which is not being utilized. In other words, by underloading the plant, the average output would have supplied the whole of the daily demand had that output been constant. In this respect the author seems to think, for obvious reasons, that it is impossible to secure, in practice, any such uniformity of load; but if it is possible to relieve the station of such peak loads by drawing the excess power from a secondary source of accumulated power supply, or by creating some form of special boosters whereby temporary high drafts for power may be made, it would mean for the Virginian Railroad plant, the running of not more than three turbo-generator sets at a time.

The problem of meeting the demands of excessive peaks that occur for short periods must be carefully considered. Assuming that such a thing is possible, the maximum demand of the generators would be reduced from 48 000 kw. to 30 000 kw. during the peak intervals. This would make it possible to shut down one generator throughout the day. It is with an idea of meeting the highest peak loads that the extra plant is made necessary. This extra plant is a measure of the increase caused by the capital charges for the running plant and incidentally for buildings and other equipment. From the data given by the author one may note that the average load for a normal day is only about 30% and on a maximum congested day not more than 43% of the full capacity of the generating plant. The effect of this low ratio and of allowing the extra plant to run idle is noticeable in the actual output in kilowatt-hours per pound of coal consumed. The station coal rate has averaged even less than $1\frac{3}{4}$ lb. per kw-hr. of net output, or about $14\frac{1}{2}$% efficiency, which is poor.

In referring to the use of pulverized fuel,* it must be said that although combustion would be better and firing easier, the fuel cannot constitute the source of energy by itself. It would be useless unless there was practically an unlimited supply and a sufficient circulation of air. It is invariably necessary in every plant to make careful calculations and provide suitable arrangements for the supply of oxygen to the combustion chamber. Perhaps the provisions made on the Virginian Railway are adequate. The author has shown all the principal merits of location and planning of the Virginian Railway power plant, in a way that should open the eyes of designers to the conditions that must be satisfied, in order that a project shall be a financial success. The choice of site and other details of planning are decidedly problems for the engineer as distinct from the designer and inventor of machinery and apparatus.

A noteworthy item concerning the transmission arrangement is the decision to secure a separate right of way intersecting with the railway only at the sub-stations.† In places where such a plan is practicable without future trouble and other public inconveniences, direct transmission routes should always be adopted, as far as possible. As a matter of fact the economy effected by reducing the transmission circuit in this project is very great.

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 21.
† *Loc. cit.*, p. 28.

The saving in 34 miles means nearly 770 tons of steel in the towers, 136 miles of transmission cables, and other incidental expenses.

The choice between direct and alternating current is a matter of the very foremost importance. This will depend on the distance to which the electric current is to be supplied and the purpose for which it is to be used. It seems to have been agreed that where there are no other conditions affecting the choice of current, the high-tension alternating system is best for larger supplies. It is also a fact that transformers for alternating current offer great advantages over those for direct current. They are stationary, require perhaps little or no attention, and can be constructed for any amount of power; and their efficiency may be as much as 96 or 98 per cent. From the point of view of economy in the electric mains or conductors, however, three-phase current is best; next is direct current; and last, alternating current. For railway traction itself three-phase motors are the best and most economical.

At present, there are about fifteen locomotives on the Virginian Railway. Assuming that they are all moving in one direction in a day of, say, 12 hours, the time headway per train would be 1 hour per locomotive. At an average speed of 20 miles per hour, this would give an actual space headway of 20 miles between any two trains. The ordinary spacing of sub-stations with a much lower voltage arrangement would be 20 miles. From Table 6,* it is found that, at present, the maximum number of trains either way (say, east-bound) is five. This would mean presumably an actual headway of 2.4 hours per train. Taking the lowest average of 14 miles per hour, the actual space-headway would be 29.6 miles, instead of 20 miles. To satisfy the latter requirements five main sub-stations would have been sufficient under ordinary conditions of close sub-station spacing; but in this case the operation is done at a very high voltage. Taking full advantage of the balancing wire system it would have been sufficient to increase the sub-station capacity and reduce the number of main sub-stations.

Referring to the main operating costs, the results given by the author are very encouraging.† Generally, by electrifying the road, only in a few items, such as cost of engines and maintenance of equipment, are considerable savings to be expected because these would be about 45% of the total expenses. A 30% saving in these items would mean nearly 14% reduction in the total operating cost; but the enormous increase in the capitalization and the very large incidental increase in the item of maintenance of way and structures (including the trolley and other contract system and the supporting structures) is the main handicap against electrification. Sometimes these larger expenses would go a long way toward neutralizing the 14% saving; but however small the savings may be, electrification is an up-to-date development and will displace other kinds of power in the future. It can perform a service beyond the possibilities of the steam engine and it can effect greater operating economies than are possible by steam propulsion, so that in the future these may be sufficient to pay a reasonably attractive return on the new capital charge incurred. This has proved to be the case in the Virginian electrification.

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 52.

† *Loc. cit.*, p. 58.

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PAPERS AND DISCUSSIONS

ANALYSIS OF ARCH DAMS BY THE TRIAL LOAD METHOD

Discussion*

By Messrs. L. J. Mensch and William Cain.

L. J. Mensch,† M. Am. Soc. C. E. (by letter).‡—The U. S. Bureau of Reclamation is to be congratulated for the advanced scientific improvements it has introduced with respect to the design of dams. It is true that this Bureau ought to be the home of concentrated study, and evidence that such is the case is most welcome.

Until about 1912 any engineer who proposed an arched dam 300 ft. high would have been considered an unreliable man and a dreamer, even if he designed it for cylindrical stresses of 30 000 lb. per sq. ft. Nothing but a full-sized gravity dam, with an additional safeguard provided by making it an arched structure, was considered proper. Although quite a large number of successful arch dams—many of them considerably less than 100 ft. in height—had been built in the United States and foreign countries since 1870, conservative engineers would not favor them because the distribution of stress in such structures appeared mysterious to them.

Since 1904, the Society has published quite a number of papers and discussions, in which many earnest workers have tried to clear up this subject. The general trend has been to solve the problem by assuming the arched structure to consist of two systems of members, namely, a horizontal system of arches fixed at the abutments and a vertical system of cantilevers fastened to the bed-rock.

The writer has shown, by the use of the elastic theory,§ that the stresses in such horizontal arches of relatively considerable thickness are much greater than those given by the cylinder formula. He also showed that the

* Discussion of the paper by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., continued from May, 1928, *Proceedings*.

† Chicago, Ill.

‡ Received by the Secretary, February 9, 1928.

§ *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 610.

assumed vertical cantilevers behave more like beams simply supported at the base of the dam rather than rigidly fixed at that point.

Other efforts have been made to explain the arch action by the same two systems of members, giving simplified approximate formulas.*

William Cain, M. Am. Soc. C. E., has treated† the thick cylindrical arch in a more general form.

These papers only showed that there are very much higher stresses than were formerly supposed in horizontal arches and cantilevers, but they did not explain the safety of many existing daring arched dam structures. It has been known for more than ten years that theoretical investigations along the old line were futile and the writer has previously pointed out‡ that, in thick sections, secondary arches are formed which are stable and which prevent the failure of the main arch. He has also shown that arches of increased thickness at the abutments are much more economical than arches of uniform thickness.

The engineers of the Bureau of Reclamation, according to the authors, also have adopted the system of horizontal arches and vertical cantilevers, but have gone farther than the previous designers by investigating the influence of the cantilevers not only in the center of the dam, but also those nearer the abutments. In addition, they have assumed secondary arches and arches of increasing depth. They deserve great credit for this advance because it finally enabled them to design, in perfect confidence and along rational lines, arched dams of such slender proportions as no engineer has dared to consider since the construction of the Upper Otay Dam in 1900. The fact is that the dams discussed by the authors show stresses of from 80 000 to 100 000 lb. per sq. ft., if computed by the common cylinder formula (neglecting cantilever action), and from 120 000 to 190 000 lb. per sq. ft. if calculated by Professor Cain's method.

The paper shows the enormous amount of labor it takes to design such a dam by the trial method, and the slow steps by which the latest perfected design was reached. It is possible, however, to obtain results with a still greater economy of material and labor by using more calculus and less arithmetic in the design.

Before showing this simpler method the writer wishes to restate the theory of arches under normal loads.

Let $A\ C\ B$ (Fig. 26) be the center line of a circular arch of a constant width of 1 ft. and a constant depth of t, subjected to water pressure of p lb. per sq. ft. If this arch were considered to be free at the abutments, but to act as a portion of a full circular cylinder subjected to uniform normal pressure, it would diminish in diameter and take a new position, A', C', B', concentric to its original position. The diminution of radius can be found by the following consideration: The shortening of the whole cylinder equals uniform stress

* "Gravity and Arch Action in Curved Dams," by Fred A. Noetzli, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1, and "The Relation Between Deflections and Stresses in Arch Dams," *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 284.

† "The Circular Arch Under Normal Loads," *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 250.

per square foot, $\frac{(p\ r)}{t}$, times length of periphery of cylinder, $(2\ r\ \pi)$, divided by modulus of elasticity, (E).

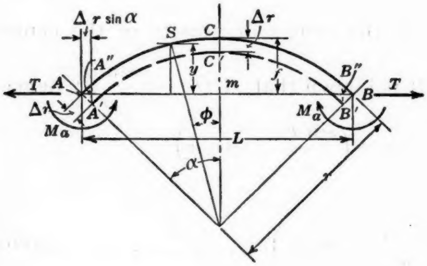

FIG. 26.

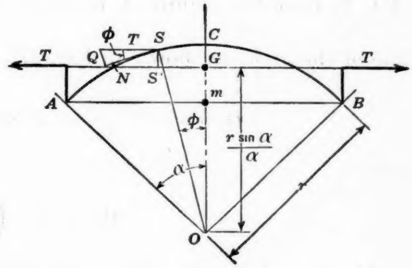

FIG. 27.

Dividing the shortening of the cylinder by $2\ \pi$ gives the shortening of the radius,

$$\Delta\ r = \frac{p\ r}{t} \times \frac{2\ r\ \pi}{E} \times \frac{1}{2\ \pi} = \frac{p\ r^2}{E\ t} \dots\dots\dots\dots\dots\dots (74)$$

From Fig. 26 it can be seen that the chord of the unrestrained arch, A', C', B', has been shortened in each half of the span an amount, $A\ A''$, equal to $\Delta\ r \sin \alpha$, or the shortening of one-half the arch span,

$$\Delta\ x = \Delta\ r \sin \alpha = \frac{p\ r^2}{E\ t} \sin \alpha \dots\dots\dots\dots\dots\dots (75)$$

This value is also given by Equation (14),*

$$\Delta\ x = \int_0^\alpha \Delta\ S \cos \phi = \int_0^\alpha \frac{p\ r}{E\ t}\ r\ d\ \phi \cos \phi = \frac{p\ r^2}{E\ t} \sin \alpha \dots\dots (76)$$

As the arch is fixed at A and B, there have to be applied horizontal forces, T, acting outward at the abutments to lengthen the chord of the arch, $A'B'$, to the original length, $A\ B$, and moments at the abutments, M_a, to keep the ends of the arch fixed in direction.

Then the moment due to the uniform shortening of the arch at any point, S, for example, is,

$$M = T\ y - M_a \dots\dots\dots\dots\dots\dots\dots\dots (77)$$

Since the angles of the tangents at A and C cannot change, then according to Equation (1),†

$$\Delta\ \theta = \int_0^\alpha \frac{M\ d\ s}{E\ I} = 0 \dots\dots\dots\dots\dots\dots (78)$$

Substituting Equation (77) in Equation (78), and transposing:

$$M_a = \frac{\int_0^\alpha y\ d\ s}{\int_0^\alpha d\ s} \times T \dots\dots\dots\dots\dots\dots (79)$$

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 81.

† *Loc. cit.*, p. 79.

The term, $\dfrac{\int_0^\alpha y\,d\,s}{\int_0^\alpha d\,s} = G\,m$, the distance of the center of gravity of the line, $A\,C\,B$, from the chord, $A\,B$, when G is the center of gravity of the center line of the arch, as shown in Fig. 27. It is known that $G\,O = r\,\dfrac{\sin\alpha}{\alpha}$; hence,

$$G\,m = r\,\frac{\sin\alpha}{\alpha} - r\cos\alpha = r\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right)$$

and,

$$M_a = T\,r\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right) \qquad (80)$$

Hence, substituting Equation (80) in Equation (77),

$$M = T\,(r\cos\phi - r\cos\alpha) - T\,r\left(\frac{\sin\alpha}{\alpha} - \cos\alpha\right) = T\,r\left(\cos\phi - \frac{\sin\alpha}{\alpha}\right)$$
$$= T \times S\,S' \qquad (81)$$

This means that it is permissible to assume T acting on a line through the center of gravity, G, of the center line of the arch (see Fig. 27).

Hence, there is only one statically unknown quantity for the solution of the effect of the shortening of the arch, namely, T, which can be found by the well known principle of least work,

$$L = \int \frac{Q^2\,d\,s}{2\left(\frac{E}{3}\right)t} + \int \frac{N^2\,d\,s}{2\,E\,t} + \int \frac{M^2\,d\,s}{E\,I} \qquad (82)$$

in which, Q, N, and M are the shear, force normal to, and moment at any section, respectively, due to the action of T only. By differentiating according to T over one-half the span only, the horizontal movement, $A''\,A$ (Fig. 26), due to the unknown force, T, can be determined as follows:

$$\frac{\delta\,L}{\delta\,T}\;A\;A'' = \frac{p\,r^2}{E\,t}\sin\alpha = 3\int_0^\alpha \frac{Q}{E\,t}\,\frac{\delta\,Q}{\delta\,T}\,d\,s + \int_0^\alpha \frac{N}{E\,t}\,\frac{\delta\,N}{\delta\,T}\,d\,s$$
$$+ \int_0^\alpha \frac{M}{E\,I}\,\frac{\delta\,M}{\delta\,T}\,d\,s \qquad (83)$$

From Fig. 27,

$$Q = T\sin\phi;\ \frac{\delta\,Q}{\delta\,T} = \sin\phi \qquad (84)$$

$$N = T\cos\phi;\ \frac{\delta\,N}{\delta\,T} = \cos\phi \qquad (85)$$

From Equation (81),

$$d\,s = r\,d\,\phi;\ \frac{\delta\,M}{\delta\,T} = r\left(\cos\phi - \frac{\sin\alpha}{\alpha}\right) \qquad (86)$$

Substituting in Equation (83), the proper values from Equations (84), (85), and (86), and integrating:

$$\frac{p\,r^2}{E\,t}\sin\alpha = \frac{T\,r}{E\,t}\left\{2\,\alpha - \frac{1}{2}\sin 2\,\alpha\right\}$$
$$+\frac{T\,r^3}{E\,I}\left\{\frac{1}{4}\sin 2\,\alpha + \frac{\alpha}{2} - \frac{\sin\alpha}{\alpha}\right\}\text{*} \quad \ldots\ldots\ldots\ldots\ldots (87)$$

If the value, j, is substituted for $\left(2\,\alpha - \frac{1}{2}\sin 2\,\alpha\right)$ and the value, k, for $\left(\frac{1}{4}\sin 2\,\alpha + \frac{\alpha}{2} - \frac{\sin\alpha}{\alpha}\right)$, Equation (87) becomes:

$$\frac{p\,r^2}{E\,t}\sin\alpha = \frac{T\,r}{E\,t}\left[j + \frac{12\,r^2}{t^2}\,k\right] \quad \ldots\ldots\ldots\ldots\ldots (88)$$

from which, T can be readily found.

If the uniform shortening is produced by a drop of temperature, t_0, or an equivalent shrinkage of concrete, the term, $\frac{p\,r^2}{E\,t}$ (which is $\Delta\,r$) must be replaced by $c\,t_0\,r$. This value, substituted in Equation (88), gives:

$$c\,t_0\,r\sin\alpha = \frac{T\,r}{E\,t}\left[j + \frac{12\,r^2}{t^2}\,k\right] \quad \ldots\ldots\ldots\ldots\ldots (89)$$

Equations (88) and (89) become more useful by introducing the rise of the arch, f (Fig. 26), instead of the radius, and, for water pressure:

$$f = r\,(1 - \cos\alpha)$$

$$T = \frac{p\,r\sin\alpha}{j + \frac{12}{(1-\cos\alpha)^2}\,\frac{f^2}{t^2}\,k} \quad \ldots\ldots\ldots\ldots\ldots (90)$$

For change of temperature:

$$T_\theta = \frac{E\,c\,t_0\,t\sin\alpha}{j + \frac{12}{(1-\cos\alpha)^2}\,\frac{f^2}{t^2}\,k} \quad \ldots\ldots\ldots\ldots\ldots (91)$$

Equations (90) and (91) are more readily solved by the use of Table 11.

The equations of T in this form show the influence of the thickness of the arch very clearly. For example, assume the ratio of rise to thickness, $\frac{f}{t} = 3$, and assume $\alpha = \frac{\pi}{6}$. Then, from Table 11,

$$T = \frac{p\,r}{1.13 \times 9 + 1.228} = \frac{p\,r}{11.398} = 0.0878\,p\,r.$$

T acts at a point, below the crown, $r - r\,\frac{\sin\alpha}{\alpha} = r - 0.95494 = 0.0451\,r.$

The rise of the arch, f, $= 0.133\,r$, and $t = \frac{f}{3} = 0.0444\,r$. Hence, the short-

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 237, Equations (10) and (11); also, p. 249.

ening of the arch in this case produces a moment at the crown equal to $p\,r \times 0.0878 \times 0.451\ r = 0.00396\ p\,r^2$; a moment at the abutment of $0.0878\ p\,r\ (0.133\ r - 0.0451\ r) = 0.00775\ p\,r^2$; a normal force at the crown of $p\,r - 0.0878\ p\,r = 0.9122\ p\,r$; and a normal force at the abutment of $p\,r - 0.0878\ p\,r \cos 30 = 0.924\ p\,r$.

TABLE 11.—TABLE OF CONSTANTS FOR VARIOUS VALUES OF α.

α.	j.	k.	$\frac{\sin \alpha}{\alpha}$.	f.	T.	T_0.
$\frac{\pi}{6}$	0.642	0.00084	0.9549	$0.133\,r$	$\frac{p\,r}{1.13\,\frac{f^2}{t^2} + 1.228}$	$\frac{E\,c\,t_0\,t}{1.13\,\frac{f^2}{t^2} + 1.228}$
$\frac{\pi}{4}$	1.0708	0.00608	0.900	$0.293\,r$	$\frac{p\,r}{1.21\,\frac{f^2}{t^2} + 1.515}$	$\frac{E\,c\,t_0\,t}{1.21\,\frac{f^2}{t^2} + 1.515}$
$\frac{\pi}{3}$	1.6614	0.0239	0.8276	$0.50\,r$	$\frac{p\,r}{1.33\,\frac{f^2}{t^2} + 1.92}$	$\frac{E\,c\,t_0\,t}{1.33\,\frac{f^2}{t^2} + 1.92}$
$\frac{\pi}{2}$	3.1416	0.1488	0.6366	$1.00\,r$	$\frac{p\,r}{1.785\,\frac{f^2}{t^2} + 3.14}$	$\frac{E\,c\,t_0\,t}{1.785\,\frac{f^2}{t^2} + 3.14}$

The eccentricity of the normal force at the crown is,

$$\frac{0.00396\ p\ r^2}{0.9122\ p\ r} = 0.00435\ r = 0.098\ t$$

and that at the abutment is,

$$\frac{0.00775\ p\ r^2}{0.924\ p\ r} = 0.0084\ r = 0.189\ t$$

While the eccentricity at the crown is less than one-tenth the thickness of the arch, the point of application of the normal force at the abutment is outside the middle third. From the equations in Table 11, it is clear that tensile stresses are produced in all arches, whether flat or steep, whenever $\frac{f}{t}$ is less than 3.

The tensile stresses are, in fact, much larger than when a fall of temperature or shrinkage of concrete is taken into consideration, as the following calculations show.

Assume in the same arch, $p = 4\,000$ lb.; $r = 270$ ft.; $f = 270\ (1 - \cos 30) = 36$ ft.; $f = \frac{t}{3} = 12$ ft.; then,

$$E\ c\ t_0 = 288 \times 10^6 \times 0.0000055 \times 12 = 19\,000 \text{ lb.}$$

for the very low drop of only 12°, and from Table 11,

$$T_0 = \frac{E\ c\ t_0\ t}{1.13\,\frac{f^2}{t^2} + 1.228} = \frac{19\,000 \times 12}{11.398} = 20\,000 \text{ lb.}$$

The moment at the crown is,

$$20\,000 \times \left(r - r\,\frac{\sin\,\alpha}{\alpha}\right) = 20\,000 \times 0.0451\ r = 243\,000 \text{ ft-lb.}$$

the moment at the abutment is,

$$20\,000 \times (f - 0.0451\ r) = 20\,000 \times 0.0882\ r = 478\,000 \text{ ft-lb.}$$

The normal force at the crown is,

$$0.9122\ p\ r - 20\,000 = 965\,000 \text{ lb.}$$

the normal force at the abutment is,

$$0.924\ p\ r - 20\,000 \cos 30 = 979\,000 \text{ lb.}$$

The eccentricity at the crown due to temperature only is,

$$\frac{243\,000}{965\,000} = 0.252 \text{ ft.} = 0.021\ t$$

the eccentricity at the abutment due to temperature only is,

$$\frac{478\,000}{979\,000} = 0.489 \text{ ft.} = 0.407\ t$$

Hence, the total eccentricity from shortening of the arch and drop of temperature at the crown $= 0.098\ t + 0.021\ t = 0.119\ t$, and at the abutment $= 0.189\ t + 0.0407\ t = 0.230\ t$.

The maximum stress (see Fig. 28) $= \dfrac{N}{0.81\ t} \times 2 = 2.48\ \dfrac{N}{t}$, or nearly $2\frac{1}{2}$ times the stress from the cylinder theory.

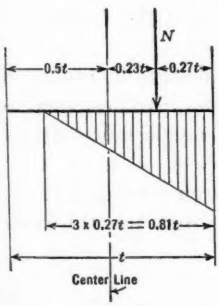

FIG. 28.

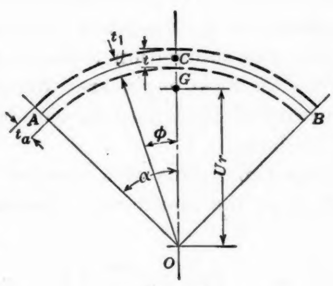

FIG. 29.

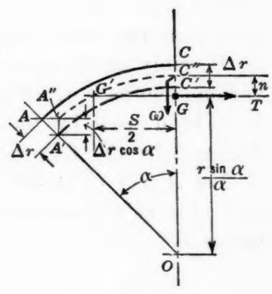

FIG. 30.

Therefore, where $\dfrac{f}{t}$ is less than 3, secondary arches are forming and economical design requires that the arch should not be of constant thickness throughout but should have its section enlarged where tensile stresses occur.

If the influence of shear is omitted from consideration,*

$$T = \frac{p\ r}{1.130\ \dfrac{f^2}{t^2} + 0.9568} = 0.0899\ p\ r$$

instead of $0.0878\ p\ r$; or T is 2% larger than when shear is considered.

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

Where (as in the Gibson Dam) $\frac{f}{t} = 1$, it is easily found that T, without consideration of shear, becomes 7% larger; but where (as in the lower part of the Bear Valley Dam), $\frac{f}{t} = 0.22$, Table 11 shows that,

$$T = \frac{p\,r}{1.13 \times 0.22^2 + 1.228} = 0.778\ p\ r$$

and from Cain's formula,

$$T = \frac{p\,r}{0.130\,\frac{f^2}{t_2} + 0.9568} = 0.989\ p\ r$$

This is a great difference and it shows that the dam could not exist without secondary arch action.

None of these formulas applies to non-reinforced arches in which tensile stresses occur, and such stresses do occur in nearly all arches (plain and reinforced), in which $\frac{f}{t}$ is less than 3. Secondary arches are then acting within such arches, having a larger rise, smaller radius, smaller thickness, and also a different pressure than the original arch. The water pressure will be different because the water will intrude in the cracks at the extrados near the abutments.

It is known that the eccentricity of the normal force at the abutments is about twice as large as that at the crown, and it is a mystery to the writer why arched dams are not designed in the first place in the shape of the future secondary arches, with enlargements at the dangerous sections.

Possibly one reason is that the summation method of designing arches of various depths is too laborious for most engineers. Secondary arches, such as have been used by the Bureau of Reclamation, can be much more simply designed by the following method.

Let $A\ C\ B$ (Fig. 29) be the center line of a circular arch having a thickness of t at the crown and t_a at the abutment. Assume that the thickness varies as:

$$\frac{1}{t_1} = \frac{1}{t}\left(1 - K\,\frac{\sin\phi}{\sin\alpha}\right) \ldots\ldots\ldots\ldots\ldots\ldots (92)$$

in which, K is a constant; I = the moment of inertia of the crown; I_a = the moment of inertia at the abutment; and I_l = the moment of inertia at any point.

For all practical purposes:

$$\frac{1}{I_1} = \frac{1}{I}\left(1 - L\,\frac{\sin\phi}{\sin\alpha}\right) \ldots\ldots\ldots\ldots\ldots\ldots (93)$$

in which, L is another constant. When the thickness of the abutment is $1\frac{1}{2}$ times the thickness at the crown, K may be assumed as $\frac{1}{3}$ and the corresponding L as $\frac{3}{4}$. When the thickness at the abutment is 2, assume that $K = \frac{1}{2}$ and

the corresponding $L = \frac{9}{10}$. The writer has checked this assumption in numerous cases with the summation method and found only very slight variation in results.

In order to solve the arch (Fig. 29) by the same method as for an arch of constant depth, it is necessary to know the new value, $A\,A'' = \Delta\,x$ (see Fig. 30) and the new line in which T is acting, which does not now go through the center of gravity of the arch but through the center of gravity of the so-called elastic weight, $\frac{d\,s}{I_1}$. By the use of Equation (76) modified by Equation (92),

$$\Delta\,x = \int_0^\alpha \Delta\,s \cos\phi = \int_0^\alpha \frac{p\,r}{E\,t}\left(1 - K\,\frac{\sin\phi}{\sin\alpha}\right) r\,d\,\phi \cos\phi$$

$$= \frac{p\,r^2}{E\,t} \sin\alpha \left(1 - \frac{K}{2}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (94)$$

The new distance, $G\,O$ (Fig. 30), is,

$$\frac{\int_0^\alpha \frac{y\,d\,s}{I_1}}{\int_0^\alpha \frac{d\,s}{I_1}} = \frac{\frac{r^2}{I}\sin\alpha\left(1 - \frac{L}{2}\right)}{s\,\frac{r}{I}\left[\alpha - L\,\frac{(1 - \cos\alpha)}{\sin\alpha}\right]} = \frac{\left(1 - \frac{L}{2}\right) r \sin\alpha}{\alpha - L\,\frac{(1 - \cos\alpha)}{\sin\alpha}} = U\,r \,.\,(95)$$

Assigning actual values to $\sin\alpha$, the corresponding values of U (See Equation (95)) for values of $t_a = 1\frac{1}{2}\,t$, and $2\,t$, respectively, are as given in Table 12.

TABLE 12.—TABLES OF CONSTANTS FOR CORRESPONDING VALUES OF α.

$a =$.	$t_a = 1\frac{1}{2}\,t$.				$t_a = 2\,t$.			
	U.	j.	k.	T.	U.	j.	k.	T.
$\frac{\pi}{6}$	0.968	0.5020	0.00038	$\frac{pr}{0.64\,\frac{f^2}{t^2} + 1.20}$	0.970	0.4459	0.000268	$\frac{pr}{0.48\,\frac{f^2}{t^2} + 1.19}$
$\frac{\pi}{4}$	0.931	0.8619	0.002733	$\frac{pr}{0.65\,\frac{f^2}{t^2} + 1.46}$	0.943	0.7542	0.00186	$\frac{pr}{0.491\,\frac{f^2}{t^2} + 1.40}$
$\frac{\pi}{3}$	0.880	1.3086	0.01 0611	$\frac{pr}{0.707\,\frac{f^2}{t^2} + 1.81}$	0.903	1.1322	0.006968	$\frac{pr}{0.515\,\frac{f^2}{t^2} + 1.74}$
$\frac{\pi}{2}$	0.762	......	....	....	0.819	......	....	....

The moment at any point, due to the shortening of the arch, is:

$$M = T\,(r\cos\phi - U\,r) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (96)$$

In the least work equation,

$$L = \int \frac{Q^2\,d\,s}{2\left(\frac{E}{3}\right) t_1} + \int \frac{N^2\,d\,s}{2\,E\,t_1} + \int \frac{M\,d\,s}{E\,I_1} \ldots\ldots\ldots\ldots (97)$$

Let,

$$Q = T \sin \phi\,; \quad \frac{\delta Q}{\delta T} = \sin \phi \ldots\ldots\ldots\ldots\ldots\ldots (98)$$

$$N = T \cos \phi\,; \quad \frac{\delta N}{\delta T} = \cos \phi \ldots\ldots\ldots\ldots\ldots\ldots (99)$$

$$M = T\, r\, (\cos \phi - U); \quad \frac{\delta M}{\delta T} = r\, (\cos \phi - U) \ldots\ldots\ldots (100)$$

$$\frac{d\, s}{t_1} = \frac{r\, d\, \phi}{t} \left(1 - K \frac{\sin \phi}{\sin \alpha}\right); \text{ and } \frac{d\, s}{I_1} = \frac{r\, d\, \phi}{I}\, 1 - L \frac{\sin \phi}{\sin \alpha} \ldots (101)$$

Substituting Equation (94) in Equation (97) and differentiating according to T, $\frac{\delta L}{\delta T} = \Delta\, x$, is the movement of one abutment, and when integrated over one-half the span, the result is,

$$\Delta\, x = \frac{p\, r^2}{E\, t} \sin \alpha \left(1 - \frac{K}{2}\right) = \frac{3\, T\, r}{E\, t} \int_0^{\alpha} \sin^2 \phi \left(1 - K \frac{\sin \phi}{\sin \alpha}\right) d\, \phi$$
$$+ \frac{T\, r}{E\, t} \int_0^{\alpha} \cos^2 \left(1 - K \frac{\sin \phi}{\sin \alpha}\right) d\, \phi + \frac{T\, r^3}{E\, I} \int_0^{\alpha} (\cos \phi - U)^2$$
$$\left(1 - L \frac{\sin \phi}{\sin \alpha}\right) d\, \phi \ldots\ldots\ldots\ldots\ldots\ldots (102)$$

This reduces again to the form:

$$\frac{p\, r^2}{E\, t} \sin \alpha \left(1 - \frac{K}{2}\right) = \frac{T\, r}{E\, t} \left\{ j + \frac{12\, r^2}{t^2\, k} \right\} \ldots\ldots\ldots (103)$$

The values of j, k, and T for various values of α, are as given in Table 12. The change of temperature, T_0, is obtained by substituting the expression, $\Delta\, x = c\, t_0\, r \sin \alpha$, in Equation (97) to get an expression corresponding to Equation (103). Then, for values of $\alpha = \frac{\pi}{6}$ and $t_a = 1\frac{1}{2}\, t$,

$$T_0 = \frac{E\, c\, t_0\, t}{\left(1 - \frac{K}{2}\right) 0.64 \frac{f^2}{t^2} + 1.20} \ldots\ldots\ldots\ldots (104)$$

A study of Table 12 shows that the expression for T will hardly change for values of α smaller than $\frac{\pi}{6}$, and it will be quite as permissible to use the same formula for smaller values of α as for $\frac{\pi}{6}$. The authors advise using Fig. 4* for estimating the change of length, due to a drop of temperature. Presumably this includes the influence of shrinkage also. The Bureau of Reclamation has probably the most complete information on this subject, and a statement issuing from its office, containing observations on which this diagram is based, would be highly welcome. What influence has the gradual widening of the arch, near the abutment, on the stresses of arch in Fig. 29?

From Table 12:

$$T = \frac{p\, r}{0.64 \times 3^2 + 1.20} = 0.144\, p\, r$$

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 66.

From Equation (104),

$$T_0 = \frac{19\ 000 \times 12}{\left\{ \left(1 - \frac{1}{6}\right) 0.64 \times 3^2 \right\} + 1.20} = 394\ 000 \text{ lb.}$$

N at the crown equals,

$$p\ r - 0.144\ p\ r - 39\ 400 = 0.856\ \ p\ r - 39\ 400 \text{ lb.} = 885\ 000 \text{ lb.}$$

N at the abutment equals,

$$p\ r - (0.144\ p\ r + 39\ 400) \cos 30 = 0.8772\ p\ r - 34\ 000 \text{ lb.} = 913\ 000 \text{ lb.}$$

M at the crown equals,

$$(T + T_0)\ (1 - U)\ r = 195\ 000 \times 0.032\ r = 1\ 670\ 000 \text{ ft-lb.}$$

M at the abutment equals,

$$(T + T_0)\ (U\ r - r \cos 30) = 195\ 000 \times 0.101\ r = 5\ 350\ 000 \text{ ft-lb.}$$

Eccentricity at the crown equals,

$$\frac{1\ 670\ 000}{885\ 000} = 1.89 \text{ ft.} = 0.1575\ t$$

Eccentricity at the abutment equals,

$$\frac{5\ 350\ 000}{913\ 000} = 5.86 \text{ ft.} = 0.325\ t_a$$

The maximum stress at the abutment (see Fig. 28) is,

$$S = \frac{N}{3\ \left(\frac{1}{2}\ t_a - 0.325\ t_a\right)} \times 2 = 3.82\ \frac{N}{t_a} \qquad \text{.......... (105)}$$

or, notwithstanding the enlargement of the arch, the stresses, due to the shortening of the arch and a drop of temperature, become larger than before, which shows that the selection of a shape of the arch, according to Fig. 29, is not always a fortunate one.

In the authors' method of designing it is very important to find the deflection of the arches at several points. This can be done simply by the following consideration: In Fig. 30, assume $A\ C$ to be the center line of the arch and, after uniform compression, the arch moved upward into the position, $A''\ C''$. Then apply a force, T, in order to bring A'' back to A.

In this position the crown of the arch, C'', is already lowered a distance,

$$\Delta\ r - \Delta\ r \cos \alpha = \frac{p\ r^2}{E\ t}\ (1 - \cos \alpha) \qquad \text{............. (106)}$$

The deflection can be found* by assuming the left half of the arch to be fixed at A'', and the force, T, acting at G and a fictive force, w, acting at the crown, then to differentiate Equation (82) according to w, making $w = 0$ afterward:

$$Q = T \sin \phi + w \cos \phi;\ \frac{\delta\ Q}{\delta\ w} = \cos \phi \qquad \text{............. (107)}$$

$$N = -\ T \cos \phi\ + w \sin \phi; \frac{\delta\ N}{\delta\ w} = \sin \phi \qquad \text{........... (108)}$$

$$M = -\ T\ r\ \left(\cos \phi - \frac{\sin \alpha}{\alpha}\right) + w\ r \sin \phi;\ \ \frac{\delta\ M}{\delta\ w} = r \sin \phi \qquad \text{.... (109)}$$

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 236.

The vertical movement of the crown of the arch in position, $A'' C'' B''$ (Fig. 30):

$$\frac{\delta L}{\delta w} = 3 \int_0^\alpha \frac{T\,r}{E\,t} \sin\phi \cos\phi \, d\,\phi - \int_0^\alpha \frac{T\,r}{E\,t} \sin\phi \cos\phi \, d\,\phi$$

$$- \int_0^\alpha \frac{T\,r^3}{E\,I} \left(\cos\phi - \frac{\sin\alpha}{\alpha}\right) \sin\phi \, d\,\phi \ldots\ldots\ldots\ldots (110)$$

Integrating and adding $\Delta\, r\,(1 - \cos\alpha)$, the total deflection of the crown is,

$$\Delta\, f = \frac{p\,r^2}{E t}\,(1 - \cos\alpha)$$

$$+ \frac{T\,r}{E\,t} \left\{ \sin^2\alpha + \frac{12\,r^2}{t^2} \left[\frac{\sin\alpha}{\alpha}\,(1 - \cos\alpha) - \frac{\sin^2\alpha}{2} \right] \right\} \ldots\ldots (111)$$

The vertical displacement of an arch with a depth increasing from the crown to the abutment is found, similarly, by using Equations (92) and (93) in Equation (82).

The vertical displacement of the point, G', in relation to C'' is easily found from the well known equation,

$$\Delta\, y = \int \frac{M\,x\,d\,s}{E\,I} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (112)$$

Consider $G'\,C''$ as y in the well known equation, $y = \frac{4\,n}{S^2}\,(S\,x - x^2)$, with the origin at G': $M = T\,y$ for this section, and, with normal force and shear neglected,

$$\Delta\, y = \int_0^{\frac{s}{2}} - \frac{T\,y\,x\,ds}{E\,I} = \frac{4\,n}{S^2}\,\frac{1}{E\,I} \int_0^{\frac{s}{2}} (S\,x^2 - x^3)\,d\,x = \frac{5}{48}\,\frac{T\,s\,n}{E\,I} \ldots\ldots (113)$$

This point and the tangents at the crown and abutment being known, it should not be much trouble to draw the deflection for the whole arch.

The writer has not mentioned Poisson's ratio because most engineers find it safer to forget it. Positive proof of its beneficial influence is still sadly lacking. The so-called curved beam action has been ignored, because theory and tests show that, for the extraordinary case where $\frac{r}{t} = 1$, the stresses are influenced 25%; for the very rare case of $\frac{r}{t} = 4$, they are influenced 9%, and for $\frac{r}{t} = 10$ (still a very thick arch), they are influenced to the amount of 4 per cent.

The stresses in the lower arches of a dam are very high, due to the shortening of the arch and a drop of temperature and shrinkage. Very daring dams are standing, however, and an explanation is urgently needed.

In seeking help from the cantilevers near the crown of the arch, it is found that the cantilevers themselves, according to theory, are stressed to the limit in nearly all cases.

The authors sought help from the cantilevers near the abutments. They assumed that the cantilevers at the crown take a very much smaller load

than was formerly ascribed to them by most engineers and, in fact, they considered the horizontal arches to have a horizontal and vertical abutment at the end. This amounts, in effect, to the assumption of a horizontal arch of a smaller span.

In large arch dams as a rule, it is uneconomical to assume the arch rings to be affected only by water pressure. If the weight of the arch ring is also considered, it will be found that the most economical design is obtained by inclined arches, the inclination being given by the water pressure per square foot to the weight of the inclined ring per linear foot.* The importance of this statement will be seen in the following demonstration.

Fig. 31 shows a section through the crown of the Dam No. II, mentioned by the authors. At Elevation 5 280, the water pressure is 10 400 lb. per sq. ft., the weight of the arch ring is 5 200 lb. per lin. ft., and the ratio of water pressure and weight of ring is exactly 2 to 1.

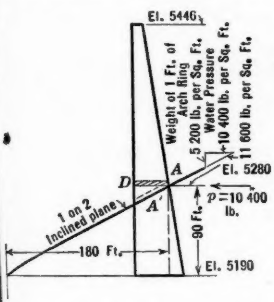

FIG. 31.

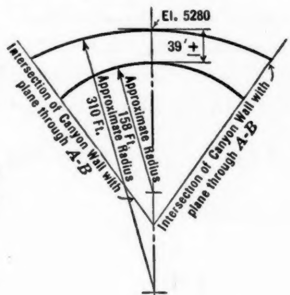

FIG. 32.

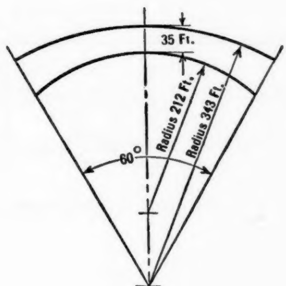

FIG. 33.

Draw Line $A\ B$ with an inclination of 1 on 2, and lay, through Line $A\ B$, a plane perpendicular to the plane of the paper. This plane will intersect the dam as shown in Fig. 32, while the horizontal section $A\ D$, is shown in Fig. 33. It is apparent that the radius of the inclined arch is very much shorter than the horizontal arch and will, by the principle of least work, support a much greater part of the water pressure than the horizontal arch. Of course, the inclined arch has to carry a larger load, the resultant of both vertical and horizontal load, namely, 11 600 lb. per lin. ft. Its thickness is also about 10% less than that of the horizontal arch, but it is 10% deeper and the transmission of the water pressure to the assumed arch will be different from that shown in Fig. 31. The water pressure will center around A' and will be transmitted by shear and friction. The assertion that $A\ B$ ought to be inclined in exactly this ratio should be modified to admit that there might be cases, depending on the configuration of the canyon, where another inclination will be of greater advantage.

The writer still maintains that it would be of much greater satisfaction to most engineers if cantilever action were entirely ignored with respect to the assistance cantilevers would give to horizontal or inclined arches, and he recommends starting the design of an arched dam by properly designing the

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 253.

inclined arches, and assuming them to carry their own weight and water pressure, these inclined arches to be affected by a shortening of the arch and a drop of temperature.

The writer is quite convinced that the arches are, in fact, very much stronger than given by the standard design for the following reasons.

It is well known that a concrete beam, when tested to destruction, is much stronger than given by the formula,

$$M = S \frac{b\,d^2}{6} \qquad \text{(114)}$$

In fact, S, computed from bending tests, is generally twice as high as the estimated tensile strength of the concrete. The reason is that the stresses do not follow Hooke's law; and instead of a straight line distribution of stresses, there is, especially near the ultimate load, a parabolic distribution of some power greater than 2.

Tests on eccentrically loaded columns also show a much greater strength, than the old straight line formula indicates.*

Assuming a distribution of stress based on a cubic parabola (see Fig. 34), tensile stresses are found to act in a rectangular section, when the eccentricity of loading is larger than $\frac{1}{10}\,t$ $\left(\text{not } \frac{1}{6}\,t \text{ as according to the old theory}\right)$; but the maximum stresses are then only $1.33\,\frac{N}{t}$.

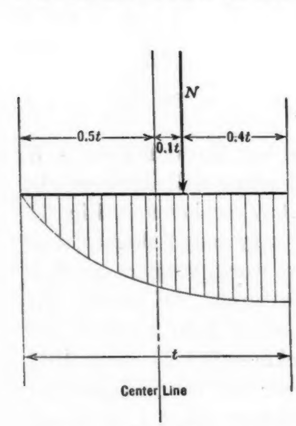

FIG. 34.

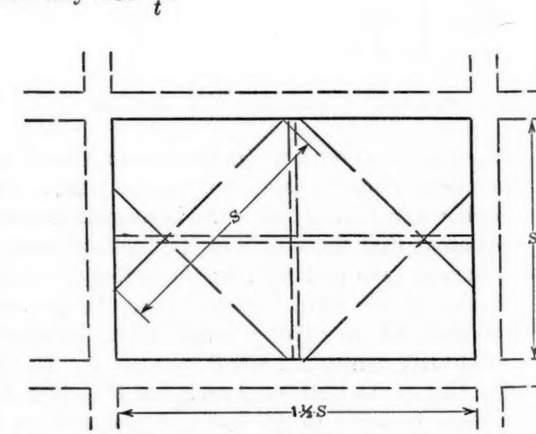

FIG. 35.

Considère† compared a 16-in. column, loaded with an eccentricity of 4 in., with an identical column axially loaded. The latter failed at 3 600 lb. per sq. in., while the former failed (according to the old theory) at 4 950 lb. per sq. in. As the concrete strength was only 3 600 lb. per sq. in., the column actually carried $\frac{4\,950}{3\,600} = 1.37$ times the load the old theory would lead one to

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1671.

† Rept., Commision du Beton Armé, 1907.

expect, and this result agrees, within 1%, with the cubic parabola stress distribution. Careful comparisons with many other tests* also check the writer's theory.

Table 13 lists the maximum stresses for various concrete sections having no tensile strength (on account of joints), e being the eccentricity and t, the depth of a rectangular section, 1 ft. wide.

TABLE 13.—MAXIMUM STRESSES IN COLUMNS FOR VARIOUS VALUES OF e.

$\frac{e}{t}$.	$\frac{N}{t}$ according to cubic parabola stress distribution.	$\frac{N}{t}$ according to straight line formula.
0.017	1.06	1.104
0.030	1.10	1.18
0.051	1.17	1.306
0.084	1.28	1.504
0.100	1.33	1.60
0.120	1.41	1.72
0.140	1.48	1.84
0.160	1.57	1.96
0.180	1.67	2.08
0.200	1.78	2.22
0.260	2.22	2.78
0.340	3.33	4.16
0.420	6.67	8.35
0.480	20.00	33.33

Table 13 shows that the stresses are in most cases 20% smaller than was formerly believed. The arches are, besides, much stronger than the common elastic theory indicates for the reason that the modulus of elasticity does not remain constant. At the abutments, the stresses first reach high values, and the modulus of elasticity becomes rapidly smaller. This means that the deformation of the arch near the abutments is relatively larger than the theory assumes. It has the effect of lowering the point of application of T, so that the greatest moment (which occurs always at the abutments), is decreased and the moment at the crown is increased. A similar effect is found in continuous beams, and this induced the Joint Committee on Specifications for Concrete and Reinforced Concrete to recommend that continuous beams be fully loaded and fully fixed at the support in order to be designed for a moment of $\frac{p\,l^2}{16}$ $\left(\text{instead of } \frac{p\,l^2}{12} \text{ at the support, and } \frac{p\,l^2}{24} \text{ in the center}\right)$.

It is very probable that for this reason alone the arches are 33% stronger than the elastic theory teaches. It is high time that the colleges discard the archaic hypotheses, on which they build their theories. Tests ought to be made with the proper spirit of finding the true laws of the strength of materials, regardless of tradition, and tests to destruction on small model dams would soon demonstrate to the most conservative engineers the enormous strength of arched dams.

The problem of the arched dam has a corollary in the slab, supported on four beams used in building construction. In Fig. 35 such a slab is shown

* *Beton und Eisen*, 1916, p. 56; and, also, *Forschungsarbeiten*, Verein Deutscher Ingenieure, Heft 166–169, 1914.

with a center strip in two directions. In all building ordinances in Germany, it is required that the division of the load for each system of strips throughout the center slab shall be assumed in the inverse ratio of the fourth power of their respective spans, for the only reason that the center strips have the same deflection at the center of the slab, when uniformly loaded in that ratio. Even in the United States building ordinances require a division according to the third power of the ratio of the spans. In France, however, it was found by tests, nearly 30 years ago, that the division of the loads ought to be according to the second power of the ratio of the spans. The explanation was not forthcoming for a long time, until an Italian engineer (Danusso) demonstrated that diagonal strips, near the corners, as shown in Fig. 34, greatly reduce the span in the long direction. The authors have demonstrated a very similar fact for the arched dam.

WILLIAM CAIN,* M. AM. SOC. C. E. (by letter).†—The writer agrees with the authors that, for the complete solution of the arch dam, the whole dam must be considered, which thus involves the shape of the canyon. Friction between the horizontal arches and between the vertical cantilevers is ignored and a series of proportional loads on these two elements are, in turn, assumed until one series is found to give the same radial deflections at the same points for both elements. Of course, for an exact solution, the friction referred to should be considered as it plays a very important rôle in transferring stress toward the abutments, particularly across cracks (either vertical or horizontal) and it doubtless increases in going from the crown to the abutments where the deflection is zero. This friction or shear is not of much importance in a dam, without cracks, that is fixed (encastrée) at the base; but for the usual arch dam, that is not fixed at the base, and with cracks, it adds materially to the strength and stability of the structure. Necessarily, by the authors' tentative method of solution it has to be ignored.

With regard to the division of the normal water pressure between a horizontal arch and the vertical cantilevers crossing it at the depth considered, some general conclusions can be given. To avoid going over old ground, the writer would refer to his discussion of the paper by Fred A. Noetzli, M. Am. Soc. C. E., entitled, "Gravity and Arch Action in Curved Dams",‡ especially to Fig. 15 in connection with the analysis for the Wooling Dam. The diagram refers to a cantilever at the crown, but it equally applies to one anywhere if subjected to its proper proportional water load, the arch deflection being computed for the point considered. On any cantilever, there is one point, K, where all the water pressure is carried by the arch and none by the cantilever. Under its proportional water load, the cantilever would bend so as to be concave down stream; but above K, it is found that, under this load, it tends to deflect more than the arch. This excess deflection is resisted by the arch, which exerts a horizontal pull on it acting up stream with a consequent equal reaction from the cantilever. The part of the cantilever above K thus bends so as to become concave up stream. The smaller the down-stream deflection of the arch, the

* Prof. Emeritus, Univ. of North Carolina, Chapel Hill, N. C.

† Received by the Secretary, April 14, 1928.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 73–79.

greater its pull on the cantilever; so that for points above K, if the deflection of the arch decreases in going from the crown, C, to the abutment, A, then the up-stream pulls on the cantilever should increase from C to A, provided all the cantilevers are of the same stiffness, as for a level dam site with cantilevers of the same size and height, that are not cracked or that have similar cracks.

The same conclusions hold when the dam site is a canyon, since then the stiffness of the cantilevers increases in going from C to A. If the deflection of the arch, decreases irregularly in going from C to A, or if the cracks in the cantilever are not similar, then the variation in the load on the cantilever will likewise be irregular.

Next, consider points below K on any cantilever on which the total horizontal water pressure is p lb. per sq. ft.

Let p' = the unit normal pressure carried by the arch; and,
p'' = the unit normal pressure carried by the cantilever; so that
$p' + p'' = p$, a constant.

First, assume that, for a level dam site, the radial deflections of the arch diminish from the crown, C, to the abutment, A, where the deflection is zero. Then, since the deflections of the arch and any cantilever, at the same point, must be equal, it follows that the deflections of the cantilevers must decrease from a maximum at the crown to zero at the abutment. Further, suppose the cantilevers to be alike and without cracks or with exactly similar cracks similarly placed, so that each one will deflect the same amount for the same load; then, in order that the deflection shall decrease from some value at C, to zero at A, the unit normal pressure p'', acting on a cantilever must decrease in going from C to A. Consequently, since $p' + p'' = p$ (a constant), then p', the unit normal pressure on the arch, must increase in going from C to A.

If, however, it is assumed that the more central cantilevers are the only ones weakened by cracks, then p'' can be much less than before at C, and it may actually increase from C to a point near the abutment. The fixing of the cantilever by the side walls at the abutment will prevent any deflection, no matter what the value of p'', and this influence will extend some distance out, so that, it is possible for p'' (and therefore, p') to remain constant from C to A, or for p'' to increase, in which case, p' decreases in going from C to A.

In the case of the canyon, Fig. 1 (c),* the cantilevers becoming shorter in going from C to A, increase in stiffness and their deflections decrease for the same load, so that it is possible for p'' to remain constant throughout (considering the fixation caused by the side walls). In that case, p' is constant, or p'' can actually increase and p' can decrease in going from C to A, particularly if cracks occur in some of the cantilevers in the middle portion. In no case is it necessary that p' should be zero at A, as seems to be assumed by the authors.

All the deflections were assumed to be down stream and to decrease in going from C to A. In the arches at the crest, near the crown of the Stevenson Creek Experimental Dam, the deflection was down stream at the crown, but up stream nearer the abutment, the neutral line forming a reversed curve.

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 62.

Evidently, the loading on the arch was such as to produce this curve and thus to prove an exception to what would naturally be expected. From all that precedes, it is seen that the proportionate loads sustained by arch and cantilever cannot be predicted. Such loads may increase or decrease in going from crown to abutment, or they may be nearly uniform, depending on the shape of the canyon and the cracks, or portions subjected to tension, particularly in the cantilevers.

In Fig. 5,* referring to Dam No. 1, the load over most of the horizontal arch was uniform. In this dam, tension was supposed to be provided for; but in the other dams, where the material everywhere was supposed to be incapable of taking tension, the proportional loadings were very variable. Nearly all arch dams are not fixed (encastrée) at the base in that no provision is made for the inevitable tension at that point for dams supposed to be fixed at the base. The authors have properly excluded the parts of the sections where tension is found, which occurs when the resultant on the section falls outside its middle third, so that only the part of a section in compression is considered in the computation.

The cantilever loads must include the weight of the concrete above the section and that of the water vertically over the up-stream face, the negative load due to the arches near the top, and the assumed proportional part of the water load on the part of the cantilever above the section considered. When the resultant of such loads cuts the section outside its middle third, say, at a distance, c, from the nearest edge, then only a length, $3c$, of the section, measured from the edge, will be supposed to be in compression; also, the neutral axis will be taken at the center of this compressed portion, and the moments and moment of inertia must be taken about this center. For sections where the resultant falls within the middle third, the moments are taken, as usual, about the center of the section.

After these moments at the various sections have been computed, the deflections of the cantilever at various points are ascertained. This can be effected by calculation, but the writer prefers a neat graphical construction,† since the deflections can be read off to 0.001 in. with practical accuracy.

The authors do not make the usual erroneous assumption that the cantilever is fixed at the base. It is simply supposed to be supported at the base and to have there a vertical tangent to the neutral line. It can only be supposed to be fixed at the base when sufficient tension can be exerted, and this is ignored in the computation.

For very thick dams, the deflection due to shear should be added to that corresponding to moments. This is easily computed from the well-known formula for the deflection in feet, h ft., above the lower section, Fig. 36,

$$\Delta = \frac{1.2}{144\ G} \int_0^h \frac{S}{a}\, d\, x$$

corresponding to a parabolic distribution of shear on a cross-section of area, a.

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 68.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 74.

In this formula,

S = total horizontal shear due to loads acting on the cantilever, x ft. above the lower section;

a = area of horizontal section where shear is S;

G = modulus of rigidity, in pounds per square inch.*

The integral gives the value of the shaded area, so that the deflection, in inches, is, for a trapezoidal section,

$$\Delta = \frac{0.1}{G} \frac{1}{2} \left(\frac{S_1}{a_1} + \frac{S_2}{a_2} \right) h$$

If preferred, the shaded area can be written, $\frac{S_0}{a_0} h$, in which, S_0 and a_0 represent the shear and area, respectively, of the section of the cantilever at the distance, $\frac{h}{2}$, above the lower section. The cantilever having been divided into a number of parts, each of approximate trapezoidal section, the deflection of the top of each part with respect to its base can be computed from this formula and the sum taken for the total deflection at a given height.

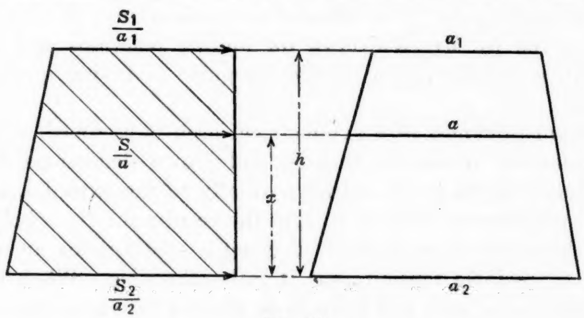

FIG. 36.

It will be seen from this discussion that the authors' method, of counting as effective only the parts of the sections of cantilevers in compression in computing moments, must necessarily lead to different results from those found on the assumption that the cantilever can resist the tension. As a matter of fact, American dams are not usually reinforced (the Wooling Dam is the only reinforced dam as far as the writer knows) and cracks are to be expected. Where they occur, it is manifestly absurd to count on tension being exerted across the cracks, or that the dam remains intact, and yet this assumption is the basis of the usual analysis. This basis, not agreeing with the facts, should be rejected for other than reinforced dams. The authors deserve thanks for fearlessly working from the facts and dealing with the actual dam with supposed openings (or cracks) where tension would occur and not with an ideal dam without cracks.

In the usual analysis, the dam is supposed to remain intact. Then, the solution is effected for the dam without weight and omitting the weight of

* In *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 100, B. F. Jakobsen, M. Am. Soc. C. E., gives reasons for taking $G = \frac{E}{2.4}$.

water vertically over it. The latter forces are to be included later. By the authors, they are included from the start, so that it is possible to find the points where tension would be exerted and the sections under compression which alone are to be regarded as effective in computing deflections and stresses.

Similar computations are to be made for cantilevers at other points than the crown and for arches at various elevations. The labor of so many hundreds of trial computations is enormous and looks prohibitive except with a large office force; so that any hints that may reduce this labor may prove welcome.

By shifting the origin to the "elastic center", the number of terms in Equations (24) to (29)* is greatly reduced and the solution more easily effected. Otherwise, in place of normal loads, equivalent vertical and horizontal loads can be taken. The solution, where such loads are taken as unit loads for symmetrical arches, has been given by Charles S. Whitney, M. Am. Soc. C. E.† Very simple formulas are derived for finding the reactions when bending only is considered, the effects of direct thrust being treated separately. The writer‡ derived very simple formulas for the reactions for both vertical and horizontal loads (including symmetrical horizontal loads) where the chances for mistake seem to be reduced to a minimum. However, the graphical method given by Mr. Whitney§ for finding reactions and deflections at any point will especially appeal to the computer as giving quickly the quantities sought.

If a uniform thickness of a symmetrical arch is assumed, it is possible to derive formulas for moments, thrusts, and crown deflections for two symmetrically placed single loads, acting normally to the extrados, and then, for any assumed continuous loading, to sum the results for the whole load. This is probably the quickest method of effecting a solution for an arch nowhere subjected to appreciable tension; but it fails otherwise. When the thickness of the arch varies, the arch will have to be divided into a number of parts and a method of summation will have to be used.‖ For the unsymmetrical arch, Melan's method may suffice.

In conclusion, the writer wishes to express his appreciation of the authors' valuable paper which is a decided step forward where light was most needed.

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, pp. 83–84.

† "Design of Symmetrical Concrete Arches," *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 931.

‡ *Loc. cit.*, p. 1030.

§ *Loc. cit.*, p. 957, and Figs. 5 and 16.

‖ See *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 515, for Mr. Jakobsen's proposed solution.

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PAPERS AND DISCUSSIONS

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STREAM FLOW IN GENERAL TERMS

Discussion*

BY MESSRS. F. THEODORE MAVIS AND MORROUGH P. O'BRIEN.

F. THEODORE MAVIS,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The author's use of the term, "friction factor",§ for the slope of the energy line is confusing since that term has long been used in referring to the measure of channel roughness, particularly in formulas for flow of fluids in pipes. The friction factor, f, in the Chezy formula, $h = f\,\frac{1}{d}\,\frac{v^2}{2\,g}$, is a function of the Reynolds number, $\frac{v\,d}{\nu}$, and the character of the conduit, and it is not the slope of the energy line, $\frac{h}{l}$. (h is the loss in energy head in a distance, l; d, the diameter of the conduit; v, the velocity; and ν, the coefficient of kinematic viscosity of the fluid, or its viscosity divided by its density.‖ Many texts refer to the slope factor in the Chezy formula and its subsequent modifications as the slope of the water surface, although this definition leads to inconsistencies which are clarified if S is defined as the slope of the total energy line.

The author states¶ that,

"Equations (10) and (11) are applicable to any invert profile and to any shape of channel section, constant or variable, open or closed, flowing freely or under pressure. * * * From Equations (10) and (11), the energy gradient * * * can be computed for any given channel for an assumed Q."

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* Discussion of the paper by Melvin D. Casler, M. Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Urbana, Ill.

‡ Received by the Secretary, March 14, 1928.

§ *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 97.

‖ See Walker, Lewis, and McAdams, "Principles of Chemical Engineering" (McGraw-Hill Book Co.), for values of f as a function of the Reynolds number.

¶ *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 101.

These equations, which are identical, are merely expressions of Bernoulli's theorem which states that the difference between the total energy $\left(\frac{p}{w} + z + \frac{v^2}{2\,g}\right)$, at two points on a given stream line is equal to the losses of energy between those two points. If this theorem is to be applied to any variable section one must know not only the amounts of various energy losses but also the pressure and velocity distributions, in order to arrive at even an approximate solution of a given problem. With regard to the second part of the author's statement it may be said that nothing has been presented in the paper which will allow one to calculate the energy gradient and hydraulic gradient for flow in a rectangular channel of uniform section, in which the flow changes from torrential flow (less than critical depth) to ordinary turbulent flow (greater than critical depth), even if one neglects perimeter friction and assumes constant velocity in a given cross-section of the stream.

The writer does not agree with the statement made in the first part of "Criterion (3)".* namely,

> "The general criterion for selecting the correct value of d as between these two mathematical possibilities is as follows: The water will flow at such a depth, d, that the lost head, F, between any two sections will be as nearly as possible equal to the invert drop, I, in that same stretch of channel. * * *"

Except in the very special case of steady flow at constant depth in an open channel with a uniform section, there is absolutely no general relation between the slope of the energy line and the slope of the invert of the channel. Assume, for instance, that the slope of the invert of a rectangular channel increases from 1 to 10% (in the direction of flow). The slope of the energy line below the point of intersection of these grade lines may be greater or less than the slope of the line above that point, depending on whether the water is accelerated on the steeper slope or whether it is decelerated so that the depth of flow becomes greater and greater.

The last sentence of Criterion (3), "successive values of $(d + h)$ will be as nearly as possible equal to each other", is ambiguous and would easily lead to the erroneous impression that the height of a hydraulic jump is the difference between corresponding depths of flow, d, for the same total energy $(d + h)$ as shown in the author's diagrams.

The author has stressed the energy equations of one-dimensional flow and has made no mention of the impulse and momentum equations. The latter are often indispensable in arriving at quantitative values of head losses, for instance, at sudden enlargements in section or at a hydraulic jump. These equations follow directly from the fundamental law of mechanics,

$$\frac{d}{d\,t}\,(m\,v) = F \qquad (62)$$

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 102.

Since the mass of water moving between two cross-sections of a channel is,

$$m = \frac{Q\,w}{g}\,d\,t$$

and, assuming steady flow, Q is constant,

$$\frac{d\,(m\,v)}{d\,t} = \frac{Q\,w}{g}\,(v_1 - v_2) = F \qquad (63)$$

in which,

Q = rate of discharge;
w = weight of a unit volume of water;
g = acceleration of gravity;
v_1 and v_2 = velocities at Sections 1 and 2, respectively; and
F = the resultant of forces acting on Sections 1 and 2.

Finally, if b is the width of the water prism at a depth, z, below the free surface of the water and d_1 and d_2 are the depths of flow at Sections 1 and 2,

$$F = w \int_0^{d_2} z\,b\,d\,z - w \int_0^{d_1} z\,b\,d\,z = \frac{Q\,w}{g}\,(v_1 - v_2) \qquad (64)$$

For a rectangular channel of constant width, b, $\left(v_1 = \frac{Q}{b\,d_1}, v_2 = \frac{Q}{b\,d_2}\right)$, Equation (64) (dividing through by w) becomes,

$$d_2^2 - d_1^2 = \frac{2\,Q^2}{b\,g}\left(\frac{1}{d_1} - \frac{1}{d_2}\right)$$

or,

$$d_1\,d_2\,(d_1 + d_2) = \frac{2\,Q^2}{b\,g} \qquad (65)$$

Equation (65) expresses the relation between the depths, d_1 and d_2, above and below a hydraulic jump* in a rectangular channel assuming "one-dimensional" flow and neglecting friction along the walls of the channel. The energy loss due to the jump is, therefore, $\left(d_1 + \frac{v_1^2}{2\,g}\right) - \left(d_2 + \frac{v_2^2}{2\,g}\right)$.

One would infer from the treatment of the critical section and from Criterion (4)† that a critical stage must be found in a given channel before the calculations of flow referred to by the author can be made. In any given problem which is capable of solution, if the depth of flow, the discharge, and the properties of the channel are known at any point the computations referred to can be carried through for conditions of one-dimensional flow. It happens that, at a critical section, the depth of flow is determinable, but the existence of a section where the velocity passes through the "critical" is rather the exceptional than the usual case. If the velocities exceed the critical, that is, if a critical section exists, the computer will have need for other tools in addition to Equations (10) and (11).

The only difficulties in problems of one-dimensional flow are to find the magnitude of the losses. With the possible exception of shock losses at sud-

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* For data on hydraulic jump phenomena, see, Julian Hinds, M. Am. Soc. C. E., *Engineering News-Record*, 1920, v. 85, p. 1034; Böss, "Berechnung der Wasserspiegellage" (*Forschungsarbeiten*, Heft 284, V. D. I. Verlag, Berlin); Koch-Carstanjen, "Bewegung des Wassers" (Springer, Berlin); Reports of the Miami Conservancy District; Safranez, "Wechselsprung und die Energievernichtung des Wassers," *Der Bauingenieur*, December 3, 1927.

† *Proceedings*, Am. Soc. C. E., Papers and Discussions, January, 1928, p. 111.

den enlargements of section, which the writer has mentioned, the solution of these difficulties can be found only by experiment. The results of many experiments (for example, experiments on flow of water in pipes and open channels, flow through orifices and over weirs), have been generalized so that they can be used readily. Only elementary mathematical tools are necessary in solving problems of one-dimensional flow.

By no means, however, may all the problems of flow in open channels be considered to be one-dimensional flow problems even for purposes of engineering design. If the problem is one of two or three-dimensional flow the engineer will probably avoid the methods of mathematical hydrodynamics in its solution and rely almost wholly on experimental methods. Model experiments in the hydraulic laboratory should be emphasized as one of the most useful methods and one generally applicable for studying those problems of hydrodynamics and stream flow which have thus far defied mathematical analysis.

MORROUGH P. O'BRIEN,* JUN. AM. SOC. C. E. (by letter).†—Although at first glance, Mr. Casler's method of computing non-uniform flow seems to be quite simple, it is, with its several criteria, more complicated to apply than the methods of many other writers who have treated the subject, notably Flamant,‡ A. H. Gibson,§ and more recently Tylvad,‖ Schonweller,¶ Böss,** Koch,†† and Lindquist.‡‡ Of the formulas proposed probably the most general is that of Lindquist for channels of constant cross-section:

$$d\,d = \frac{1}{2}\,\frac{\zeta_n \left(\dfrac{d}{d_n}\right)^3 - \zeta}{\left(\dfrac{d}{d_c}\right)^3 - 1}\,d\,l \qquad (66)$$

This includes all the necessary criteria since d_n is the depth for normal or uniform flow, d_c is the critical depth, and ζ_n and ζ are factors depending on the slope of the energy gradient at uniform flow, and at the flow considered, respectively. The values of ζ to be used are to be obtained from the formulas:

$$\zeta = \frac{2\,g}{\sqrt[3]{R}\,M^2}$$

for the higher alternate stage, and,

$$\zeta = a + \frac{d}{\sqrt[3]{\dfrac{R\,v}{v}}}$$

for the lower alternate stage.

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* Asst. Research Engr., Hydr. Structures Laboratory, Royal Technical Univ., Stockholm, Sweden.

†Received by the Secretary, April 27, 1928.

‡ Flamant, "Hydraulique."

§ A. H. Gibson, "Hydraulics and Its Applications."

‖ K. Tylvad, "Graensvaerdier for Vandets Hastighed ved Strommende Vandbevaegelse," *Ingenioren,* January 28, 1928.

¶ G. Schonweller, "Beregning af Vandforing med frit Vandspejl," *Ingenioren,* January 28, 1928.

** Böss, "Berechnung der Wasserspiegellage," *Forschungsarbeiten,* Heft 284, V. D. I., Berlin, 1927.

†† Koch and Carstanjen, "Bewegung des Wassers," Berlin, 1926.

‡‡E. Lindquist "Anordningar för effektiv energiomvandling vid foten av överfallsdammar," Anniversary Vol., Royal Technical Univ., Stockholm, 1927.

The constants, M, a, and b, are to be found in special tables* and v is the kinematic viscosity. The differentiation between the frictional losses at the higher and lower stages of flow resulted from a study of some of Bazin's experiments, which showed that the manner of flow was essentially different at the two stages and that at the lower stage the loss is practically the same as for the towing of flat plates held parallel to the direction of motion.†

An even more general formula is that of Freytag which includes terms representing the change in the bottom width and the side slope,

$$\frac{d\,t}{d\,l} = \frac{\dfrac{v^2}{C^2\,R} \mp \dfrac{Q^2}{g\,a^3} \cdot \dfrac{\delta\,a}{\delta\,c} \times i \mp \dfrac{Q^2}{g\,a^3} \cdot \left(\dfrac{\delta\,a}{\delta\,\alpha} \times \dfrac{d\,\alpha}{d\,l} + \dfrac{\delta\,a}{\delta\,b}\dfrac{d\,b}{d\,l}\right)}{1 \mp \dfrac{Q^2}{g\,a^3} \cdot \dfrac{\delta\,a}{\delta\,d}} \quad \ldots\ldots (67)$$

in which,

α = the side slope;
b = the width of the bottom; and
z = the elevation of the water surface.

The negative sign is to be used for accelerated, and the positive sign for decelerated, flow.

Although Equations (66) and (67) appear to be far more complicated than those given by Mr. Casler, they are equally simple to apply since in a practical case the differentials could be replaced by finite increments. They have the advantage that in some cases all the variable quantities can be expressed in terms of the distance‡ and the exact equation for the water surface can be obtained by integration.§‖

The author indicates a very common error in the use of the Chezy-Eytelwein equation, namely, that the slope to be used is either the slope of the bottom or of the water surface. However, even when this slope factor is correctly taken to be the slope of the energy gradient, the question arises as to whether the same coefficients may be used in computing it for non-uniform flow. Böss¶ made a few experiments on this problem and found that for both accelerated and decelerated flow, the coefficient, n, in the Kutter formula was the same, but his experiments were few in number and were made in a very small channel. It would seem that for an equal mean velocity in the same channel, the drop in the energy line would be greater during decelerated than during accelerated flow. In his method of computation, Koch includes an impact loss for decelerated flow.

In most computations of flow, the velocity head is based on the mean velocity without making a correction for the effect of the unequal distribution of velocity. The exact expression for h is,

* E. Lindquist "Anordningar för effektiv energiomvandling vid foten av överfallsdammar," Anniversary Vol., Royal Technical Univ., Stockholm, 1927.

† E. Lindquist, "Om Modellregler eller Likformighetssatser vid Vattenbyggnadstekniska Försök," *Teknisk Tidskrift*, 1925.

‡ Flamant, "Hydraulique."

§ K. Tylvad, "Graensvaerdier for Vandets Hastighed ved Strommende Vandbevaegelse," *Ingenioren*, January 28, 1928.

‖ G. Schonweller, "Beregning af Vandforing med frit Vandspejl," *Ingenioren*, January 28, 1928.

¶ Böss, "Berechnung der Wasserspiegellage," *Forschungsarbeiten*, Heft 284, V. D. I., Berlin, 1927.

$$h = \alpha \frac{\bar{v}_m^{\,2}}{2\,g} = \frac{\int_0^A \rho\, v^3\, d\, a}{2\,g \int_0^A \rho\, v\, d\, a} = \frac{\int_0^A \rho\, v^3\, d\, a}{2\,g \cdot \rho\, \bar{v}_m\, A}\,;\ \alpha = \frac{\int_0^A v^3\, d\, a}{\bar{v}_m^{\,3}\, A} \ldots\ldots (68)$$

in which,

ρ = the density;
v = the velocity over a small element of area, $d\,a$;
A = the total area of the cross-section; and
v_m = the mean velocity.

There is some dispute as to whether the value of α is the same for both the higher and lower alternate stages of flow. The values given in Table 11, found by the graphical method of Rehbock,* indicate that it is the same in channels in which the distribution of velocity is determined by the friction at the walls and not by obstructions.

TABLE 11.—Comparison of Bazin Experiments with Those of Nikuradse.

	$Q \left(\frac{m^3}{\text{sec}}\right)$.	d (m).	$v \left(\frac{m}{s}\right)$.	$d_c\,m$.	$\frac{d}{(d+h)}$.	α.
Bazin*, Series 10..	1.236	0.265	2.318	0.356	0.497	1.161
Nikuradse†.......	7.59×10^{-3}.	0.245	0.206	0.143	0.992	1.138

* Bazin, "Recherches Hydrauliques."

† Nikuradse, "Untersuchung über die Geschwindigkeitsverteilung in turbulenten Strömungen," *Forschungsarbeiten*, Heft 281, V. D. I., Berlin.

If it is necessary that α be included, an average value of 1.15 may be used. For the Bazin experiment (Table 11), an error of about 6% would have occurred in $(d + h)$, if α had been neglected, but for the Nikuradse experiment, the error would have been inappreciable. If the flow is disturbed by sills or other obstructions, α may have a value of 2 or even more.

Although the expression for the critical depth may be obtained directly from such general formulas as those of Gibson and Freytag† from the condition of a vertical water surface, the usual method is to base it on the condition of a maximum discharge for a certain energy content, as was done by the author. The total energy passing any section in unit time is,

$$E_s = \frac{\alpha\, \rho\, Q^3}{2\, a^2} + \rho\, g\, d\, Q \ldots\ldots\ldots\ldots\ldots\ldots\ldots (69)$$

in which, the datum for the potential energy is the lowest point in the section considered. Expressing the width of the channel as $b = F\,(d)$, the general condition for the critical depth is,

$$F\,(d) = \frac{\left[\int_0^d F\,(d)\, \delta\, d\right]^3 g}{\alpha\, Q^2}$$

* Th. Rehbock, "Die Bestimmung der Lage der Energielinie bei fliessenden Gewassern mit Hilfe des Geschwindigkeitshöhen-Ausgleichwertes," *Der Bauingenieur*, 1922.

† L. Freytag, "Der Wasserabfluss in Flossgassen u. ähnlichen Gerinnen," *Forschungsarbeiten*, Heft 235, V. D. I., Berlin.

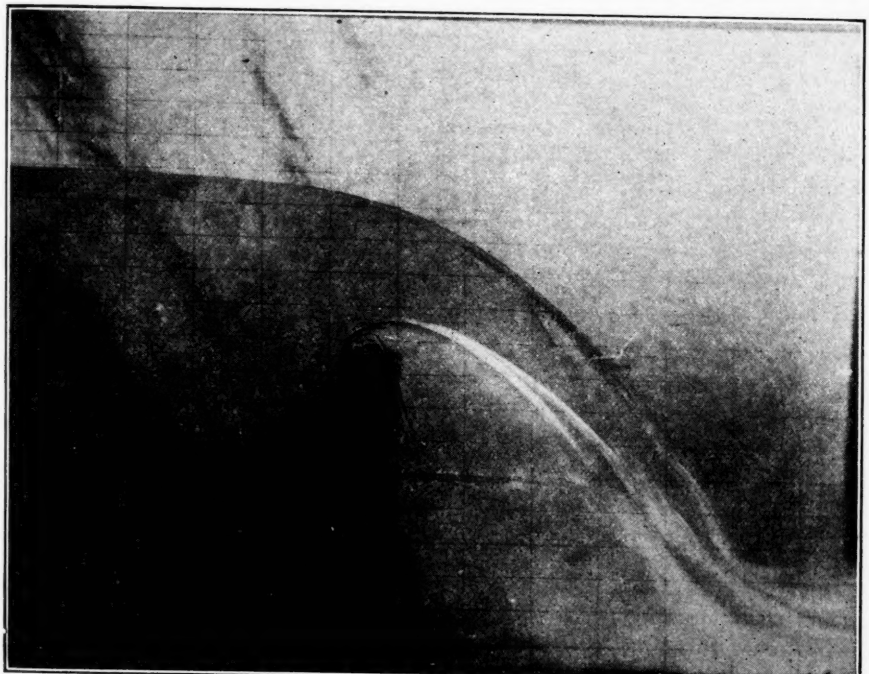

FIG. 14.—VIEW SHOWING CRITICAL DEPTH.

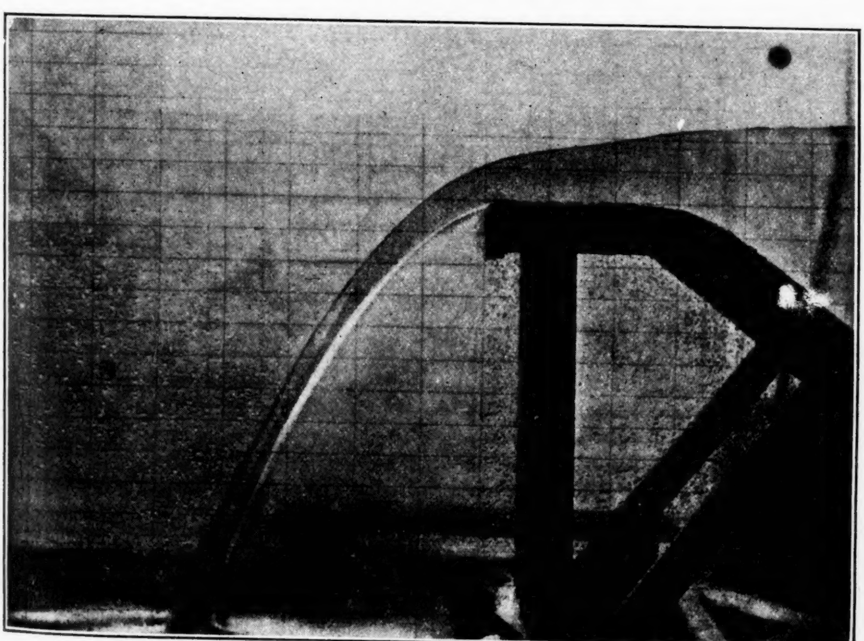

FIG. 15.—EXPERIMENT ON BROAD-CRESTED DAM.

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or,

$$b = \frac{a^3\, g}{\alpha\, Q^2}$$

For a channel of rectangular section this gives,

$$d_c = \sqrt[3]{\frac{\alpha\, Q^2}{g\, b^2}}$$

and for a parabolic section,

$$d_c = \sqrt[4]{\frac{\alpha\, Q^2}{\frac{8}{27}\, p^2\, g}}$$

in which, p is the constant of the parabola.

However, this derivation does not seem to be rigorously correct since it assumes that the potential and kinetic energy are mutually convertible and that the water surface remains horizontal in passing a controlling section. Considering any transverse section of a stream, the height of the energy line is a maximum somewhere near the center; is at nearly the same height over most of the width; and drops abruptly to the water surface at the banks. The available energy of any vertical water column is the vertical distance from the energy line to the bottom and not the vertical distance to the lowest point in the cross-section. Consequently, it would seem that, under some conditions, the critical depth might not occur at all points of a cross-section.

A very simple method of representing the relation between the height of the energy line and the quantity of flow has been developed by Koch.* From the equations for the critical depth and the flow at any depth, the relation,

$$\frac{Q}{Q_{\text{max.}}} = \sqrt{\frac{27\, \alpha_{\text{max.}}\, (1 - n)\, n^2}{4\, \alpha}} \quad \text{.................} (70)$$

may be obtained in which it is assumed that the value of $(h + d)$ is a constant of known value. The quantity, n, is the ratio, $d\ (h + d)$; $\alpha_{\text{max.}}$ and α are the correction factors used in computing h at the critical depth and the depth considered, respectively. The following tabulation gives the values of n and the ratio, $\frac{Q}{Q_{\text{max.}}}$:

n.	$\frac{Q}{Q_{\text{max.}}}$	n.	$\frac{Q}{Q_{\text{max.}}}$
0.00	0.00	0.67	1.00
0.1	0.247	0.8	0.930
0.2	0.465	0.9	0.740
0.3	0.660	0.95	0.551
0.4	0.806	1.00	0.00
0.5	0.919		

Plotting these values along a vertical line between $(d + h)$ and $d = 0$ gives a diagram which is very convenient in studying the flow through constricted sections, especially where the change in the area of the stream is very abrupt, as at bridge piers and broad, submerged weirs.

* Koch and Carstanjen, "Bewegung des Wassers," Berlin, 1926.

In the application of his method to the computation of the coefficients of discharge of weirs and orifices, the author makes several assumptions that are not correct. He assumes, for instance, that the velocity is uniform throughout the section above the crest of a sharp-crested weir; that the critical depth occurs above the crest of a weir; and that at a section, y, beyond an orifice, the thickness of the jet may be used as d_y in Equation (42).*

Table 12 gives values of the head, depth over the crest, and critical depth for the weir shown in Fig. 14, which was tested in the Hydraulic Structures Laboratory of the Royal Technical University at Stockholm, Sweden.

TABLE 12.—TEST ON WEIR AT ROYAL TECHNICAL UNIVERSITY, STOCKHOLM, SWEDEN.

H, in meters.	$Q\left(\frac{m^3}{sec}\right)$.	d_c, in meters.	d (crest), in meters.	μ.
0.05	0.021	0.035	0.04	0.636
0.15	0.110	0.135	0.122	0.636
0.25	0.242	0.180	0.22	0.649
0.40	0.425	0.2685	0.34	0 677

The critical depth is seen to be considerably less than the depth over the crest. In Figs. 14 and 15, the horizontal lines are 5 cm., and the vertical lines are 10 cm., apart. In Fig. 14, $H = 0.25$ m., $Q = 0.242$ cu. m. per sec. per meter of crest, and $d_c = 0.180$ m. Fig. 15, in which $Q = 0.080$ cu. m. per sec. per meter of crest, shows one of a series of experiments on the discharge coefficients of broad-crested dams. The critical depth, which was about 0.0863 m., occurs 14 cm. from the down-stream end of the crest. An examination of other photographs from this series of experiments showed that for this particular dam the critical depth occurs at approximately the same point for all discharges within the range of the experiments.

If the critical depth is uniquely defined by the discharge, then it is permissible to state that the depth over the crest of a weir is a critical depth. However, this critical depth is not the same as that used in open channels where the paths of the fluid particles are sensibly parallel.

There does not seem to be any lack of methods for computing non-uniform flow in conduits, but rather a lack of sufficient coefficients for computing losses due to impact and friction and data regarding the formation of jumps, bores, and other phenomena. Since Bazin's experiments there have been some additions to the available information made by American and European laboratories, but it is insufficient and even what has been done, is not widely known. This problem should be carefully studied in a laboratory by means of small-scale experiments supplemented by measurements of large-sized streams and canals. The establishment of a National hydraulic laboratory, which is opposed by some hydraulic engineers, would provide the means for the study of this and hundreds of other similar problems and would help to remove much of the uncertainty which at present occurs in nearly all hydraulic computations.

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 120.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

FOUNDATIONS AND DRAINAGE OF HIGHWAYS

Discussion*

By Messrs. Charles Terzaghi and D. P. Krynine.

Charles Terzaghi,† M. Am. Soc. C. E. (by letter).‡—In the history of every applied science it is possible to distinguish three successive stages. In the first stage, due to limited experience, the ultimate solution of the problems involved seems to be within easy reach and attainable within a few years. In the second, a broader knowledge of the empirical facts is acquired and the unsuspected complexity of the phenomena almost completely destroys one's confidence as to the possibility of discovering anything resembling simple relations or fairly valid laws. In the third stage, a separation is achieved between essential and unessential facts; and, instead of universally valid laws of Nature, modest rules are established with a small but well-known range of validity.

Mr. Rose's stimulating paper essentially deals with the first stage of the sub-grade science, in which he has played the part of a pioneer. At present, this science is passing through the worst section of the second stage, and it may be interesting to review briefly the extent to which views have changed during the seven years which have passed since Mr. Rose made his valuable field studies in the Pacific Northwest. It usually requires outsiders to discover the shortcomings in new methods of reasoning, and the outsiders were, in this case, the Russian road engineers, backed by a long and brilliant tradition in soil science. In 1924, a few years after Mr. Rose made his investigations, the Russians started to organize their own sub-grade service (Highway Research Bureau of the Central Department of Local Transport of the People's Commissariat of Ways and Communications), and tried, as a matter of course, to avail themselves of everything that others had done in this field, particularly of the American methods. They recognized at once the weakest point of past practice. Obviously, it resided in over-estimating the importance of the properties of the soil as such. They knew from experience that the same soil

* Discussion of the paper by Albert C. Rose, Assoc. M. Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Associate Prof., Foundation Eng., Mass. Inst. Tech., Cambridge, Mass.

‡ Received by the Secretary, April 19, 1928.

may behave very differently, depending on the geological, hydrographical, climatic, and topographical conditions. They knew that no soil survey can serve its purpose unless it includes a general study of the locality, the data concerning the soil proper, such as mechanical analysis, and that soil constants represented merely a part of what is needed.

In general reconnaissance work, the Russian engineers proceed from the most essential to less essential items and, according to their technique, the quality of the raw material of the soil ranks among the less essential ones. In brief, they summarily rejected what Mr. Rose presents in his paper as Point 1* of his "Summary", protesting against what they called the conception of the soil to be "a chance association of pulverized matter". This attitude may be somewhat exaggerated; yet, it contains a great deal of truth. American engineers have learned that during the last few years. Data such as those presented in Table 2† certainly looked promising at the time when they were obtained; yet, the table is followed by the statement that "the exceptions to the rule [that the linear shrinkage of the soil be the critical test for sub-grade quality] have been noted in previous papers by the writer." Since the days when these "previous papers" were published, experience has been acquired quite rapidly. The number of exceptions was found to increase alarmingly, and—imperceptibly but irresistibly—the "center of gravity" of attention shifted from the rules toward the exceptions. There are obviously many more factors which influence the behavior of the road than one could possibly suspect seven years ago, and these are by no means as easily reached as is, for instance, the linear shrinkage. To give a few examples: The reports published during the first stage of the sub-grade science simply distinguished between good, doubtful, and poor subgrades, and the readers were delighted to be offered such a clean-cut distinction. However, a more sophisticated generation started to inquire what the terms, "good, doubtful, and poor", really mean. The answer to this question was found to be rather perplexing because experience has shown that the same soil, from the road engineers' point of view, may be very good or very bad, depending on whether it is encountered in a cut or in a fill. In other localities, the order is reversed inasmuch as the transportation of the soil from the cut into the fill decidedly improves its quality. Some of the worst frost troubles on New Hampshire roads were encountered on soils that should have been excellent considering that their linear shrinkage was less than 1.

Another factor, which was not yet considered in the earlier days, was the soil profile. Mr. Rose merely dealt with the older conceptions according to which the road engineer was satisfied to investigate the nature of the material serving immediately as a foundation; that is, the sub-grade. It is what one could call a two-dimensional conception of the problem. However, within the last few years, more and more attention has been paid to the fact that every natural superficial soil deposit consists of three superimposed layers, namely, the A, B, and C horizon (top-soil, subsoil, and parent material), with very different properties, and that the behavior of the road does not merely depend on the quality of the sub-grade proper, but also to a large extent on both the

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 145.
† *Loc cit.*, p. 131.

thickness and the properties of the layers located beneath. As a consequence, the U. S. Bureau of Public Roads adopted, quite recently, the practice of combining the work of the survey party with the work of a soil scientist whose sole duty consists in exploring the soil profile and obtaining other data which are not disclosed by the laboratory test. This new policy was the inevitable outcome of the growing insight into the complex character of the sub-grade problem, and it is practically identical with what Russian engineers have done since 1925. The new policy requires the development of novel methods for expressing the results of field observations on road maps which show, not only the character of the narrow strip of land to be occupied by the road, but also the adjoining belt, together with its hydro-geological characteristics, and the stratification of the soil from the surface down to the parent material.

Similar imperceptible, but nevertheless quite revolutionary, changes took place within the last few years in the engineer's attitude toward the soil tests themselves—a change which is hinted at in the last part of the paper. When the first soil tests were completed, many years ago, each test represented a kind of a patent medicine, its causative relations to the others being practically unknown. It was believed that a single test or two was sufficient to express, for practical purposes, the character of the sub-grade. The effects of a similar belief depreciated the value of the otherwise excellent work of the Swedish Geo-Technical Commission performed in 1914-22. Since then, however, engineers have learned that the situation is by no means so simple. Soils which, for instance, have the field moisture equivalent and the linear shrinkage in common may be very different in all their other aspects; and these other aspects may have on the quality of the sub-grade an equal or more important influence than the linear shrinkage. This behavior is particularly conspicuous with soils that represent a very poor sub-grade in spite of their low linear shrinkage, as, for instance, many of the soils in Northern New England. Hence, the efforts of investigators gradually turned from the original goal of correlating the results of one single soil test with the behavior of the roads to a broad investigation of the manifold properties of the soils—of their physical meaning, their inter-relation, and their bearing on the results of the various soil tests. Thus, not more than a few years ago, it was still believed that the Rose test expressed essentially the same properties as does the centrifuge moisture equivalent. A more careful analysis of the tests, backed by a knowledge of the underlying physical facts, disclosed that the two tests have very little in common, in a physical sense. Attempts were made to establish a mathematical relation between the silt content and the moisture equivalent. At present, engineers know the reason why there cannot possibly exist anything but a very crude statistical relation between these two quantities, valid merely for the average of a great number of different test results, but certainly not for individual materials.

The first results of these broader investigations, recently published,* reflect, clearly, the utter complexity of the situation. They disclose the

* "Present Status of Subgrade Soil Testing," by Messrs. Hogentogler, Terzaghi, and Wintermyer, *Public Roads*, Vol. 9, No. 1, March, 1928.

necessity of exploring the field even more thoroughly before one can dare to decide which and how many soil tests will be required for identifying soils reliably and at a minimum amount of time and labor.

These supplementary statements by no means curtail the merits of this paper. It undoubtedly has a lasting value as a lucid presentation of the early tentative efforts to solve the sub-grade problem, and of the opinions which served as guides. The test that bears Mr. Rose's name still ranks to-day among the simplified soil tests that deserve serious consideration.

D. P. KRYNINE,* M. AM. SOC. C. E. (by letter).†—The writer's intention is to treat only three specific details in Mr. Rose's paper.

First.—He believes that the distortion of the soil is not only vertical, but also horizontal. Inasmuch as the greatest part of the horizontal forces are counterbalanced, the visible influence of a bad sub-grade is mostly, but not always, equivalent to the action of great negative vertical forces only, such as in the case of swelling.

Second.—The author states‡ that field moisture equivalent is indicative of the shrinkage value, which in turn is important for determining the value of the soil. The content of clay by mechanical analysis, according to the author, may be considered equal to the field moisture equivalent. In other words, if the sand particles in a soil are separated from the clay, and the clay content is determined, there is a criterion for judgment of the quality of the soil as sub-grade. In reality, this is not so. The mechanical analysis and the application of soil maps in connection with Feret's tri-axial diagram seem to be insufficient for the purpose.

Third.—The author cites§ the example of a 2-in. Warrenite wearing surface on a 2-in. new stone course over a 6-in. old macadam road. He states that this surface "was not heavy enough to withstand the traffic under the existing adverse soil conditions and the inadequate surface drainage and sub-drainage". The old macadam had naturally worked in the same conditions of sub-grade and drainage. Therefore, there may be two basic cases: (*a*) before the construction of the Warrenite the old macadam had been in good condition, and hence the deterioration of the Warrenite is not due to the bad sub-grade; (*b*) before the construction of the Warrenite the old macadam had also suffered from the sub-grade distortion, in which case the question arises as to why the new Warrenite surface has been constructed over a base that was known to be in bad condition; naturally, there may be a third case (*c*) in which the traffic on the Warrenite has become greater than it had been on the macadam. If this is so, one may be led to the conclusion that a bad sub-grade can be tolerated unless the traffic attains a certain volume; and this circumstance is to be investigated.

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† Received by the Secretary, April 27, 1928.

‡ *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 125.

§ *Loc. cit.*, p. 135.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

DESIGN AND CONSTRUCTION OF CONCRETE PAVEMENTS

Discussion*

By D. P. Krynine, M. Am. Soc. C. E.

D. P. Krynine,† M. Am. Soc. C. E. (by letter).‡—Concrete slabs may be designed by one of two methods: First, by making more or less logical assumptions as to the conditions; and, second, by actually attempting to determine the conditions under which the slab will act. In the first case, the assumptions made are conditional; that is, they are true of a given loading that acts under given conditions. Generally speaking, the results thus obtained do not reflect the actual conditions. They are conservative and more or less suitable for practical purposes. In the second case, the conditions that may be expected in a specific slab are investigated and the stresses that may be expected in that particular slab are determined for any given set of conditions. The work of H. M. Westergaard, M. Am. Soc. C. E., mentioned by the author§ seems to be an enormous step beyond the first method and toward the second; but the writer believes that there is room for still further development.

The coefficient, k, which is assumed to be proportional to the deflection of the slab,‖ does not actually possess such a proportionality. The same may be said of the coefficient applied in the design of railway track, which has been used by Zimmermann, Winkler, and many other theoretical investigators of railway track. If k were proportional to the deflection, the soil on which the slab is constructed, would follow Hooke's law, which is not the case. As a certain approximation, naturally, k may be admitted to be proportional to the deflection of the slab; but it must be kept in mind that this assumption may become a source of certain errors. These errors may be either insignificant or rather great, but their degree of importance is ignored in every case.

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* Discussion of the paper by Clifford Older, M. Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Prof. of Highway Eng., Moscow Superior Technical School and Moscow Inst. of Transportation Eng., Moscow, Union of the Socialistic Soviet Republics.

‡ Received by the Secretary, April 18, 1928.

§ *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 147.

‖ *Loc. cit.*, p. 148.

This coefficient depends on the changing state of the sub-grade; therefore, the experimental method of determining it, recommended by Professor Westergaard, is not exempt from the possibility of error. Mr. Older takes this circumstance into account. He is completely right in stating* that "the deflection of a corner or edge under load at night may be several times as great as under the same load applied during the day"; and, further, that "research directed toward the determination of effective values of k under various conditions would be of great importance".

The author proposes to record the depression of small, specially constructed slabs under loads.† The writer believes that it is first advisable to make a preliminary study of the influence of different dimensions on slabs with the same load and bearing area. The investigations of Terzaghi, Goldbeck, Bijls, von Emperger, and others, show that such influence exists in the case of a slab that is uniformly loaded. The value of k must depend on the spacing of transverse joints in a highway slab. If this value, k, is determined in the case of a certain slab constructed on a given soil, the result will probably differ from that obtained in the case of a smaller or a larger slab on the same soil, the thickness of the slab in both cases being equal.

The author assumes that the loaded area is circular. In reality, it is more or less elliptical. Dr. R. Schenck gives five types of tire impressions on the metaling of the road.‡ If the length of the impression parallel to the axis of the highway is $2\,\xi$, and its width, $2\,\eta$, the ratio, $\frac{\xi}{\eta}$, is about 3. His findings are listed in Table 4.

TABLE 4.—STRESSES IN SLABS ACCORDING TO SCHENCK.

(Changed from metric units approximately.)

Tires.	Dimensions, in inches.	Load in pounds.	Area of impression, in square inches.	Specific pressure, in pounds per square inch.
Solid tires	40 by 4⅛	3 100	16.12	191
Fulda—Giants	38½ by 5⅛	3 100	23.56	130
High-pressure	37⅛ by 6	3 100	25.42	121
High-pressure	32½ by 4¾	1 300	19.69	67
Low-pressure	34 by 6¼	1 300	28.67	46

If the total wheel load, P, equals 10 000 lb., and the radii, a, of the circles over which the wheel load is assumed to be uniformly distributed, equals 6 in. at the corners and 8 in. at the edge, the areas of the circles would be 113 sq. in. and 201 sq. in., respectively, and the pressures on 1 sq. in. would be 89 and 50 lb. In other words, Professor Westergaard's specific pressures correspond only to the last lines of Table 4.

The author assumes that the wheel load is uniformly distributed over the area of the tire impression. This is the case for pneumatic tires, but the

* *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 152.

† *Loc. cit.*, p. 150.

‡ "Die Kraftwagenstrasse," Charlottenburg, Germany, 1925, p. 29.

specific pressure of the solid tires seems to be from 1.75 to 2.1 times more in the center of the impression than on its contour.* Assuming it to be twice as much in the center as it is on the contour, then for the solid tires given in Table 4, the specific pressure in the center of the tire must be $191 \times 2 = 382$ lb. per sq. in.

Since the action of the wheel load is not static but dynamic (assuming the impact coefficient as 2), the specific pressure in the middle of a solid rubber tire would attain the enormous figure of $382 \times 2 = 784$ lb. per sq. in., instead of 89 or 50 lb.

It would seem that the slab, under the action of such pressures as 784 lb. per sq. in., must break into pieces; in reality, it does not. The writer believes the reason is that the method proposed by Professor Westergaard does not result in determining the actual stresses. It is only a step—an enormous step indeed—toward discovering the real stresses; that is, his method is conditional. Therefore, the writer thinks that until Professor Westergaard publishes the results of further investigations, it is too early to put this theoretical knowledge into practice.

All branches of civil engineering connected with soils and soil mechanics (and slab design is among them) are now passing through a period similar to that passed through by chemistry when replacing alchemy. New scientific methods and new ideas were then introduced into science in a short lapse of time. The same occurs now in the science of engineering foundations, into which new principles have been introduced in the last years, principally by Charles Terzaghi, M. Am. Soc. C. E. New principles of soil mechanics must be more extensively used in all allied branches of Civil Engineering and among others, in the design of highway slabs. The writer wishes to emphasize that the work of Professor Westergaard and Mr. Older is an enormous step forward, but that further advances must be made.

The development of a new engineering soil science is not yet complete. The civil engineer in practice prefers to use the mathematical method; but this method is not unique in the foundation and highway engineer's profession. Certainly, applied mechanics, hydraulics, and other fundamental technical sciences are based on mathematics. However, it is possible to apply other scientific methods which until now, have been unknown. The ancient Romans, without knowing differential and integral calculus, constructed many buildings that have lasted to the present time. Furthermore, as Professor Terzaghi states, in medicine there are methods which, without being mathematical, are at the same time scientific.† The writer does not mean to say that a civil engineer must apply medical methods, but merely indicates the possibility of applying other methods than mathematical ones. Even in applying mathematical methods, foundation and highway engineers have not progressed far enough; or at least not as far as it is possible to go. For instance, highway slabs are designed with the assumption that the loads are static. Assume that the load is P, and the impact coefficient, m, so that the acting value of the

* Schenck, p. 22; Neumann, "Der neuzeitliche Strassenbau," Berlin, 1927, p. 36; and other German engineering works.

† "On the Methods and Possibilities of Road Soil Investigations," by Charles Terzaghi, *Proceedings*, Sixth Meeting of Highway Research Board, 1926.

applied force is $m\,P$. Then, suppose that under the static action of the force, $m\,P$, a deflection, y $(= y_0)$, is produced. The deformation takes place during a certain period of time, so that y is a function of t. Let Fig. 4 represent the curve, $y = f\,(t)$. Its shape is of no importance. Suppose that, for producing the deflection, y_0, the time, $t = t_0$, is necessary. If the dynamic load, $m\,P$, acts on the slab a very short time, $t_1 < t_0$, and the deflection during this time attains the value of y_1 (Fig. 4). The slab is a system of material points which, after removing the force, $m\,P$, remains under the influence of the forces of inertia, and the deflection attains a certain value, y'_1. In any case, $y'_1 < y_0$; in other words, the action of the moving load, $m\,P$, is equal to the static action of a load, $n\,P$, in which, $n < m$. Naturally, the stresses in the slab correspond also to the static action of the load, $n\,P$. In this manner, there are two impact coefficients: m, which is the impact coefficient of external forces; and n, the impact coefficient of stresses. Similar ideas have been expressed in modern bridge engineering.*

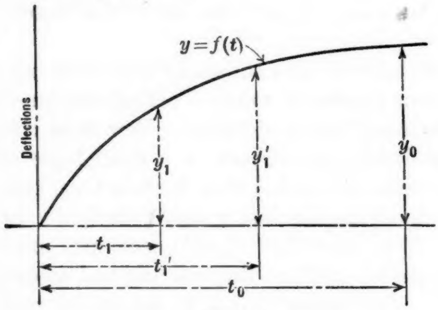

FIG. 4.

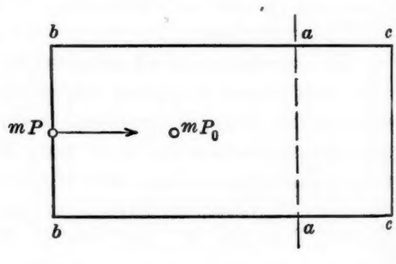

FIG. 5.—PLAN OF HIGHWAY SLAB.

The writer thinks that in the matter of designing slabs, not only the statics, but the dynamics of deformations must be taken into account. Fig. 5 represents a slab limited by two transverse joints, *b-b* and *c-c*. If the load, $m\,P$, is moving from the joint, *b-b*, to the joint, *c-c*, in the direction indicated by the arrow, the transverse section, *a-a*, receives a part of the effect produced by it, in amounts varying with the position of the moving load ($m\,P$, $m\,P_0$, etc.). A deflection is produced in the section, *a-a*, which reaches its maximum value when the load passes directly over the section. In this manner, an accumulation of deflections occurs at any section during a certain time and this, also, is a dynamic process.

In the last ten or fifteen years, highway engineers have become accustomed to dealing with colloidal matter in soils; that is, with very small particles in space. Perhaps, in the matter of designing highway slabs they will sometimes be obliged to deal with very small intervals of time. The writer thinks that further work must be done in this branch. Something similar has already been partly effected in Germany.† It is to be expected that American engineers with their wonderful facilities for investigation will not leave the slab dynamics without study, and that new works on the subject will soon appear.

* See the works of the Russian Professor N. S. Streletzky, and several German bridge engineers.

† See Schenck, p. 16, "Investigations of the Superior Technical School of Berlin."

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE ROLE OF THE CIVIL ENGINEER IN POWER DEVELOPMENT

Discussion*

By J. A. Sargent, Assoc. M. Am. Soc. C. E.

J. A. Sargent,† Assoc. M. Am. Soc. C. E. (by letter).‡—The writer is pleased to note that in his reference to the text for his "sermon", Mr. McConnell concurs§ with the belief of other wise men in assuming that, "will he, nill he", the civil engineer is perforce more or less of a missionary—although perhaps he sometimes (by force of circumstances) becomes a somewhat profane servant of The Lord. The writer has known some few such missionaries who in times of epidemic or pestilence following famine and war, have qualified, also, as medical missionaries, rendering first aid to the afflicted and spiritual consolation to the severely wounded and the dying—both on the battlefields of the World War and in the great famine and pestilential zones that rode on the heels of that war. In these rôles, the civil engineer, profane or non-profane, has often played worthy parts, and some of these parts have at times been staged in remote zones—and in the development of power.

The economic conditions and industrial requirements of any given country must influence the outlook and points of view, not only of the engineer, but also of the constructive financiers who back him and his work with the "hard cash" necessary to carry on. Without this backing, most engineers have learned that no rôle worthy the name can be played in modern power development.

The civil engineer, as well as the mechanical, electrical, architectural, and "what-not" engineer, is learning that in connection with the development of modern power projects, either for steam-electric or hydro-electric power, he must first of all play the rôle of a compiler of salient facts from original sources in the field, or his efforts are wasted. The wildernesses of South America and other "far-away countries" are dotted with projects that failed, because this rôle was either not played, or not successfully played.

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* Discussion of the paper by I. W. McConnell, M. Am. Soc. C. E., continued from March, 1928, *Proceedings*.

† With Dwight P. Robinson & Co., Inc., New York, N. Y.

‡ Received by the Secretary, April 25, 1928.

§ *Proceedings*, Am. Soc. C. E., January, 1928, Papers and Discussions, p. 155.

In many cases, examinations of surveys, selections and set-ups, estimates and mightily well-executed detailed designs, have been made by well-known engineers in wildernesses of vast potential wealth. Reports have remained shelved and dormant, not because of the lack of the potential and undeveloped wealth, but because of the failure to judge the right time-period when constructively minded financiers believe the day has arrived to warrant them to dare to risk their money. Although all life is a bit of a gamble, the many starts and shut-downs of power projects in rich but backward wilderness countries point the salient truth that, unless he wishes to waste his time, the engineer must play the rôle of a wise judge before he attempts to initiate a modern power project. He must be sure that he is addressing receptive-minded and financially capable backers, no matter how valuable may be the results of his investigations at original sources for power. Furthermore, he must be reasonably sure that the time is ripe for investing capital in his project, or "his cake"—although perfectly mixed—is "so much dough".

For example, during 1927, in a naturally rich and well populated country (not more than 10 000 miles from China), the writer had the experience of making a reconnaissance, where, beginning about 1906, a French engineer of some little international fame (who may be called De Couvrier, because that was not his name) had initiated such a reconnaissance, preliminary surveys, estimates, designs, plus far-reaching and seemingly practical investigations of the market for selling power.

After learning from the natives of the country, who lived near the various dam sites, that De Couvrier had spent several years in that region, the writer searched for a few weeks and finally discovered several reports. De Couvrier, so his several reports showed, had spent a large part of his time between 1906 and 1914, making set-up after set-up of his power project. His exceedingly painstaking search for accurate data at original sources, showed that during this period this man had gone to the most extreme pains in a jungle country, had cheerfully undergone years of hardships, and had made permanent friends with the "indignes" who still remembered him in a land where the heavy torrential rains of the terrific rainy season wash out the memories of many deaths. In fine, most memories are short-lived in a jungle country, and it is a tribute of no mean worth that the natives remembered this French engineer as a friend and adviser.

De Couvrier had discovered that, emanating from the subterranean drainages of three world-renowned mountains that are covered by perpetual snow, a continuous source for hydro-electric power development had existed for centuries, and within easy and practical limits for transmission to a large neighboring city where power was badly needed.

Came the great war. De Couvrier had not yet been able to finance and build his power project. He went to France and sometime during the war, he "went west." The writer was unable to learn in what service or where and when he had died; but was told "he died in service". De Couvrier was the kind who would do just that.

This little tale, it is believed, contains the elements of both tragedy and farce, although the writer disclaims all attempts at levity, irony, or the trite

effort to point out an obvious moral. The resultant facts are ironical enough in themselves without labeling.

While De Couvrier (so the records showed), from 1906 to 1914, was toiling assiduously with inadequate financial backing, as with the labors of Hercules, to turn his visioned "may be into a shall be", to furnish commercial and industrial power to a growing and fast modernizing city of less than 1 000 000 and more than 500 000 souls—other engineers, backed with "financial-power-upon-the-hour", were reconnoitering and finding other potential water powers, in another part of the same zone and also within practical transmission distance of the city.

The entire possible zone was not fully reconnoitered during the pre-war period. Before De Couvrier could find the necessary "hard cash" to finance and build upon his ideas, the others "got there first" with the most dollars. They builded the first units and—although it seems that the aggregate power which they first installed and transmitted to the city cost more than De Couvrier's plan would have, and although the project as built, when compared to that of De Couvrier, produced less power at maximum cost—the first units that were built served the city. The local "sabios" (sages) were loud in their praises of the builders of the project, "as is".

Nearly twenty years after this Frenchman got his "big idea" came another man who sensed that within this power zone, something valuable was going to waste—just as De Couvrier sensed the same thing from 1906 to 1914. In order to avoid being needlessly personal, let this man be known as "Mr. Ruaney" and say that he came from Philadelphia in the rôle of a tea merchant. His name is not Ruaney and he does not hail from Philadelphia, although the scene of the play was less than "10 000 miles away from China".

After looking over the zone, conferring with certain of his friends and financial associates, Mr. Ruaney ordered a reconnaissance made of the same project where De Couvrier had toiled for eight years—before he went away to die "somewhere in France".

Certain of Mr. Ruaney's friends and advisers, among them gentlemen interested in the organization that had "got there first with the most dollars", had builded the first power units (this organization is now serving the growing city with high-priced power) advised Mr. Ruaney that he was wasting his time and money. Mr. Ruaney being free born and possessing a mind of his own, persisted in his investigations. It turned out later that not one of the men who had advised Mr. Ruaney that he was wasting his time and money, had ever been near De Couvrier's power site; but they had taken the advice of others. These others, also, had not visited the project at the original sources where De Couvrier had toiled eight years in vain.

The results of the secondary investigations instigated by Mr. Ruaney showed that, in so far as physical conditions and potential supply of power is concerned, De Couvrier (making allowance for changes in pre-war labor and plant installation costs), had been conservative to a degree. More potential power than De Couvrier estimated is available and, even now, at a reasonable construction cost. There the matter stands—for the present in abeyance.

The question arises: Did De Couvrier fail or did those others who got there first and with the most dollars succeed? Point your own moral.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

GENERAL CONTRACT SYSTEM VERSUS SEGREGATED CONTRACTS

Discussion*

By Messrs. W. A. Starrett, Robert Ridgway, R. E. Bakenhus, and Wilson T. Ballard.

W. A. Starrett,† M. Am. Soc. C. E.—This paper addresses itself to the whole broad field of contracting, and comment, herewith given, refers only to that highly specialized branch generally known as large, metropolitan office buildings. Where such structures are under consideration, it has been the speaker's experience that the divided contract is most detrimental, and the unified, all-embracing contract is the only practical solution for the type of work in question.

It is true that certain architects, particularly those in the West, carry on, with considerable success, the function of the general contractor; but in doing so, they are, in effect, contractors and are organized as a general contractor is organized. The risks to the owner may or may not be pointed out to him, but the responsibility must be obvious, and he evidently calculates accordingly.

Contrary to popular belief, contracting includes very little engineering work. If it were an engineering problem at all, it would be called business engineering, but the only point of contact between the two is that a knowledge of engineering is of value to a contractor but is in no wise essential, and it is likewise measurably true that wide experience in engineering in no way assists in considering the practical problems with which the contractor is confronted.

Successful contracting is based on wide experience and it is a most complex procedure in the field of tall, metropolitan buildings. As Mr. Christie has stated,‡ the activity is almost wholly a managerial and co-ordinating

* Discussion of the paper by Ward P. Christie, Assoc. M. Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Vice-Pres., Starrett Bros., Inc., New York, N. Y.

‡ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 397.

function, but it also demands a going organization that is accustomed to "teaming" together, that understands the policies and precedents involved in the decisions and activities of the principals of the contracting company. The custom of the trade has a bearing. Contracting has grown up around sub-contracting, and it is beside the point to aver that it were better if this were not so. The whole trend of the industry is that the sub-contractor looks to the contractor for his contracts, his payment, his protection against injustice, and, most particularly, the skill of management which will enable him to get in, complete his work, and get out with a profit, not jeopardized by the risks of interference. From this situation has grown a relationship in which the best sub-contractors depend very measurably on the fair treatment and experience of the principal contractor. With one principal contractor a given sub-contract may be regarded as a hazard and risk, whereas exactly the same specified work may be considered without hazard if the general contract is handled by another and more favorably considered general contractor.

In spite of all the care that is given to predetermination and pre-planning, modern buildings are changeable and changing, even while under construction. The contracting system that gives this change the greatest elasticity without exposing the owner to disadvantageous claims is the best one. It is to be borne in mind that the desire for change almost universally arises out of the needs of the owner, seldom out of the needs of the sub-contractor. An owner represented by an architect on a highly subdivided contract is at a disadvantage when this situation arises. On the other hand, a sub-contractor depending on the general contractor with whom he may be doing business year in and year out, considers the question of changes and substitutions more equitably. Sub-contractors naturally look with suspicion on owners who have never built before and will possibly never build again. They fear that if they are thrown into direct contact, such owners may prove to be either capricious or arbitrary in their dealings. Generally, owners have no standing with the sub-contractors in their dealings and properly may be regarded with apprehension as an enterprise is undertaken.

Whatever is stated herein refers especially to private contracting. It is recognized that the Government civil agencies may not have the elasticity to deal with building problems that a private owner has. It may be expedient, apart from the mere practical question involved, for a Government to adopt any of a number of systems.

However, there seems no real justification for the separation of contracts where large, complicated structures are concerned and where the small fee paid to a capable general contractor for his services is so sure to be returned many-fold by the economies of service and administration that he contributes to the operation.

ROBERT RIDGWAY,* PAST-PRESIDENT, AM. SOC. C. E.—The speaker is an engineer and not a general contractor. As Chief Engineer of the Board of Transportation of the City of New York, he has to do with the construction of

* Chf. Engr., Board of Transportation, City of New York, New York, N. Y.

rapid transit subways. The subway construction contracts may be regarded as examples of general contracts. The large construction contracts include all the items of work that make up the complete structure such as excavation, concrete, water-proofing, and steel. They are let to the lowest bidder who can qualify, and he is made responsible for all the work to be done under the contract. He is the general contractor. The construction contracts do not include track, station finish, or equipment, which call for work of special character, which work is done under separate contracts.

The subway work is generally divided into units of from ½ to ¾ mile in length, and of a value of from \$4 000 000 to \$8 000 000. It is believed by the Board of Transportation that contracts of this size are the most advantageous because they are large enough to attract responsible concerns and not too large to restrict competition. There are exceptions to this rule where the conditions require it. For example, the under-river tunnels are let in units that sometimes cost from \$10 000 000 to \$15 000 000, or more. The largest single contract let to date is the tunnel under the East River at Fulton Street. Including its land connections, the bid price is approximately \$22 000 000.

It does not seem practicable to segregate these contracts; that is, to make separate contracts for the excavation, concrete, steel, and other items. If that were done the engineer would have to be the general superintendent in lieu of a general contractor, and there would be all sorts of interference between contractors, with resulting confusion, delay, and claims. Since they are public contracts, the Board of Transportation cannot follow the practice, common in private work, of choosing a number of qualified contractors and limiting the bidding to them. The work is open to general competition which means that the best qualified contractor is not always the low bidder, although the Board has been very fortunate in getting contractors of the character that it has. The Board has laid down the rule that the successful bidder must show liquid assets available for the job to the amount of 10% of his bid. This is to insure that the work will be carried on without the delay that always results from a lack of sufficient capital. The rule is a good one, and has the effect of eliminating those who are not financially able to carry the burden of the work. On only a few occasions, have contracts been awarded to other than the low bidder, and then only because the low bidder did not qualify.

A public contract is a rigid instrument and after it is entered into cannot be modified without an enormous amount of effort. Having this in mind the unit price form has been adopted for the subway contracts because that form provides flexibility and is believed to be fairer to both sides. A subway contract is a complicated one at the best. Not only must it take care of the building of the structure, but also of the underpinning of adjacent buildings and the restoration or reconstruction of all sub-surface utilities that exist under the busy streets of the city.

In other words, the street must be completely rebuilt, and provision must be made for paying the contractor for all this complicated work. Those who have had experience in this class of work know that it is almost impossible to predict what will be found under the surface of the city streets. The ratio between quantities of earth and rock may be different from what is antici-

pated, and the quantities of the other items may be increased or diminished. This means that the contractor should balance his bid so as to provide for such contingencies. It is believed that the lump-sum form of contract would not be as fair because of the uncertainties.

Perhaps the speaker has strayed rather far away from the subject of this excellent paper, but he wants to point out that on the Board of Transportation work the construction contracts are general contracts. If the segregated type of contract were used the engineer would have to act as the general superintendent or the general contractor, and, generally, he is not trained along those lines. It seems better to continue with the system now in use and have the trained contractor do the work instead of the engineers doing it by day's labor or by segregated contracts.

There are times, however, when one wonders whether a trial of another method would not be warranted as, for example, when a technical contractor goes to Court and wins a large sum of money, not on equity, but because a comma is misplaced in the contract. That money might have been saved if the work had been done by day's labor under the direction of the engineer. On the whole, however, the contract system is believed to be better.

R. E. Bakenhus,* M. Am. Soc. C. E.—The speaker's experience has been entirely in the State and Government service and, since 1901, with the Corps of Civil Engineers of the Navy, largely in connection with the construction work of the Bureau of Yards and Docks. It has covered all phases of contract work from the standpoint of the owner and the engineer; the owner in this case being the Federal Government. It has also been the speaker's good fortune to have had much experience in connection with the construction of public works by direct employment of labor and by the method of individual contract, thus placing the officer in charge in the position of the general contractor.

It has been, therefore, very interesting to note the author's views in regard to general contracts. There is no question that the building business in general could not get along without the general contractor; he is a very necessary part of it.

With any given project to be undertaken it seems necessary to make an individual analysis of the situation. There are a number of cases in which a general contract does not meet the situation best; perhaps they are only a few, a small minority of cases. One or two examples, of opposite extremes in regard to size, occur to the speaker. One, of course, is the Panama Canal, in which case the letting of the work by general contract was very thoroughly considered and finally rejected, and it was done entirely under the supervision of engineers, who originally had experience, of course, with day labor. The other example was the case of a small power plant which needed rehabilitation and enlargement. The appropriation was so small that the work could not have been accomplished had all the items been lumped under a general contract. It therefore became necessary to make a number of contracts which dovetailed into one another and included in individual contracts such items

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as the following: (1) Foundations for boilers; (2) erection of boilers; (3) boiler settings; (4) purchase of electric generators; (5) purchase and erection of engines as a separate contract, but including the forcing of the generators on to the engine shaft; (6) purchase of pumps in open market; (7) erection of certain piping and equipment; and, (8) construction of new wing to boiler-house.

It meant much work and anxiety on the part of the engineer to handle the work in this way but it was done successfully in this instance at a large saving in cost and without so much as a question about a single item throughout the progress of the combined work. The total cost of the Panama Canal was about \$400 000 000, and that of the power plant just mentioned about \$40 000.

Another instance in which the general contract did not seem to be advisable was the construction of the U. S. Naval Armor Plant at Charleston, W. Va. The decision to proceed with the construction of the plant was reached by the Secretary of the Navy in August, 1918, during the latter part of the World War and at a time when the naval building program was in progress. Orders were issued to prepare the designs which were completed during the next few months, and then further orders were given to go ahead with the construction work. In the meantime the Armistice had been declared. At that time the markets for labor and material were in such an uncertain state that the Department did not feel that the general contractor would be able to give a fair lump-sum bid. He might either over-estimate or under-estimate the conditions as to labor or material or both. A "cost plus" contract did not seem wise because it was a rather unpopular form in the Government service at the time. Therefore, the only decision left, as recommended by the speaker, was for the Bureau of Yards and Docks itself to become the general contractor.

Such factors as the steel frame work, steel sash, roofing, and a few minor items were made the subject of individual contracts. All the other parts of the work, including excavation, concrete foundations for buildings, furnaces and machinery, brick work, railways, roads, sewers, water piping and electric installations were undertaken by the direct employment of labor. As the Navy Department had no organization at Charleston for such work, it became necessary to purchase equipment, establish a labor office, and purchase material as well as to secure transfer of materials from other points where it was no longer needed by the Government service.

The work undertaken cost more than \$10 000 000 and was completed successfully, on time, within the limits of funds available. It is hardly likely that such a result could have been accomplished at that time by any other method of construction. So far as known, it is the largest single enterprise in the line of public works construction, which the Navy Department has ever carried through as one operation.

In this discussion, the speaker has been trying only to complete the picture which has been proffered to the Society and does not in any way mean to disparage the ideas of the author.

It is very necessary to emphasize that for any project it is desirable to make a thorough study of the situation as to how it shall be carried through

and then decide what is the best means to accomplish it. In such a situation there must be taken into account not only the project itself, but also the personnel that would be available to handle it; the market condition as to labor and material; and what general contractors are available and able to handle the project under consideration. Above all, it is necessary to take into account the experience of the engineers or other personnel who would be placed in charge of the work, either under the contractor or under direct execution by the owner. In the three cases which have been mentioned by the speaker, it has been shown that there were particular reasons why the general contract was not the method adopted.

There is no question that in the majority of cases the general contractor has the experience and ability to carry through complicated or simple construction work better than it can be done by the owner or engineer. The various reasons for this have been so clearly brought out in the paper that it is not believed necessary to repeat them.

Some of the larger corporations, as well as the Government service, have construction problems which are of a continuous character, such as railway maintenance, railway sidings, roads, and telephone and telegraph communication systems, and it has been found advisable in some of these cases to organize a permanent construction force under the control of a trained official of the organization.

The usual owner or engineer of to-day, aside from the great corporations or Governmental organizations, has a great many other things to do than to take the part of a general contractor, and usually he has not the organization or the personnel that the general contractor develops and maintains with the special knowledge and experience that must be a basis of any attempt to do general contract work. Any owner or engineer who undertakes, seriously, a project in this way without previous experience, and qualified personnel, will have many snares and pitfalls to avoid, and his experience may cost him much from the most unexpected causes. However, the Engineering Profession has given many examples of successful transition into the general contracting business.

It seems well to remember that the engineer is usually charged with more than the design of structures and projects, and the preparation of drawings and specifications. Generally, he is the one who should make the analysis of the situation on which a decision may be based as to the best method of carrying the project from a status of drawings and specifications into a completed plant or structure.

Papers of this type, with comment thereon, made by the engineers representing the various sides of the question, will result in a great deal of good.

WILSON T. BALLARD,* M. AM. SOC. C. E. (by letter).†—The author has discussed a subject of great interest and importance in these days of large construction projects. He announces an effort to throw some light on the subject of the general contract and the segregated contract. He then

* Associate, The J. E. Greiner Co., Baltimore, Md.

† Received by the Secretary, April 14, 1928.

dismisses the subject of the segregated contract in a few words, by defining it and stating that certain engineering projects, by their nature, are satisfactorily handled by segregated contracts, and proceeds with a detailed discussion of the advantages of the general contract with especial emphasis on its value in building operation. Therefore it seems worth while to bring out in discussion more concerning the segregated contract and, further, to consider certain modifications of the general contract that may broaden its scope of usefulness.

Little, if any, criticism can be offered of the statement that the general contract, in practically every instance, is the better of the two forms for building construction. In the construction of most large buildings a large number of small sub-contractors are employed, many of whom are hardly more than groups of highly specialized laborers whose efforts must be properly timed and co-ordinated.

There are many kinds of projects, however, outside the field of building construction. One group of great importance includes the building of large bridges. Each big bridge project divides itself naturally into the construction of substructure and superstructure which, except in the case of arch bridges, are totally different operations and are performed by different kinds of organizations. Frequently on a bridge of any size, the contractors on the substructure and superstructure are each competent and financially responsible. If, in such a case, one or the other occupies the position of general contractor, he does not perform a service of appreciable value in supervising the work of the other. Therefore, the owner is best served by carefully drawn separate contracts awarded to contractors of proven capacity and ability, thus avoiding the payment of double profit.

There is one qualification of the foregoing that, when made, brings work such as described back into the hands of a general contractor without calling on the owner to pay a commission or fee and receiving no commensurate value therefor. This qualification requires that the contractor who assumes the work under a general contract modify his commission in such a way that it is not made to apply to the value of work done by a large subcontractor whose ability and financial responsibility are equal to or greater than his own. For instance, assume that A, a competent company on foundations, takes the general contract and sublets the steel superstructure to B, one of the large steel companies. A's contract provides that he must deliver the piers at a certain time, after which the company furnishing and erecting the steel must do its work. During this period of steel erection, A has little if anything to do and maintains practically no organization on the job. It would be unfair to the owner therefore for him to include in his fee paid him by the owner an appreciable, if any, percentage of the value of the work done by the steel company since such a charge on his part would constitute a duplication of profit paid by the owner. This applies to other than bridge construction where the work, by its nature, divides itself into two or three large and distinctly different operations.

A further point discussed* by Mr. Christie with reference to the 3 or 4% charged by the engineer or architect for supervision during construction, might well be considered. Ordinarily, supervision during construction does not contemplate co-ordination of contractors' functions, but rather is furnished in order that the owner through his agent, the engineer, is assured that all work done is according to plans and specifications. This charge, therefore, does not represent a duplication of profit or fee for general supervision, but rather is compensation to the engineer for a distinct and separate service and one that is indispensable, especially where part or all of the money being spent comes from the investing public through the medium of bankers.

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 402.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

INLETS ON SANDY COASTS
Discussion*

By A. M. Strong, Assoc. M. Am. Soc. C. E.

A. M. Strong,† Assoc. M. Am. Soc. C. E. (by letter).‡—The subject of the hydraulics of the flow of tides through narrow entrances to inlets and lagoons and the resulting movement of sand and silt is a matter of interest in many places besides those studied by the author. The principles set forth in the paper are equally applicable elsewhere and they form, therefore, a valuable addition to the literature on the subject. Along the coast of Southern California the matter of creating and maintaining quiet anchorages for pleasure and fishing boats is becoming of considerable interest. In this district an additional factor consisting of the local geology must be taken into consideration because it is largely the controlling factor in the location and character of the entrance channels to the inlets and lagoons.

The California coast, from the region of Santa Barbara southward, has been subject to extensive block-faulting, in the course of which the various blocks have been tilted and depressed or elevated varying amounts. The shore line along the elevated blocks is rocky, with perpendicular bluffs. In some places elevated terraces or old shore lines form a shelf between the top of the bluffs and the foot of the hills; in other places, the hills come directly down to the sea. Along the sunken blocks the shore is more or less marshy, with sandy beaches, allowing the formation of lagoons back of a line of sand dunes. Some of these lagoons are of sufficient size to maintain permanent tidal inlets, but many of them are only open to the sea during and after heavy storms. Anchorages and still water for boating can be secured by deepening and protecting the natural channels to the larger inlets and by dredging the smaller ones and creating permanent artificial channels to them.

These lagoons are narrow, with the long axis parallel with the coast. They are separated from the sea by a narrow sandspit or belt of sand dunes and are

* Discussion of the paper by Earl I. Brown, M. Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Cons. Engr., Los Angeles, Calif.

‡ Received by the Secretary, April 9, 1928.

from 12 to 20 ft. deep. The natural entrance channels are at one end of the lagoon at the foot of the hills or bluff forming the edge of the adjoining fault block. The channel makes a sharp curve just inside the coast line, with the convex side of the curve in a rocky formation. The concave side of the curve is formed by the end of the sandspit. The hydraulics of this curved channel is that described* by H. C. Ripley, M. Am. Soc. C. E. The resulting increased scour on the convex side of the curve against the resistant formation prevents the migration of the channel or the formation of an inner bar. There seems to be no effect on the outer bar, other than the prevention of migration.

Fig. 10 shows one of these California lagoons and entrance channels. Prior to the construction of the jetty the entrance channel made a sharper curve well inside, and the discharge was at right angles to the shore line. The location of the jetty has tended to straighten the channel, but has also caused an extensive deposit of sand, in the form of an inner bar, making it difficult to maintain sufficient depth for navigation.

The prevailing direction of the littoral drift along this coast is to the east and the location of the entrance channel to Newport Bay at the east end of the lagoon is in accordance with the usual theories in regard to its effect. At Anaheim Bay, a few miles to the west, the entrance channel is alongside a rock formation at the west end of the lagoon. The channel forms

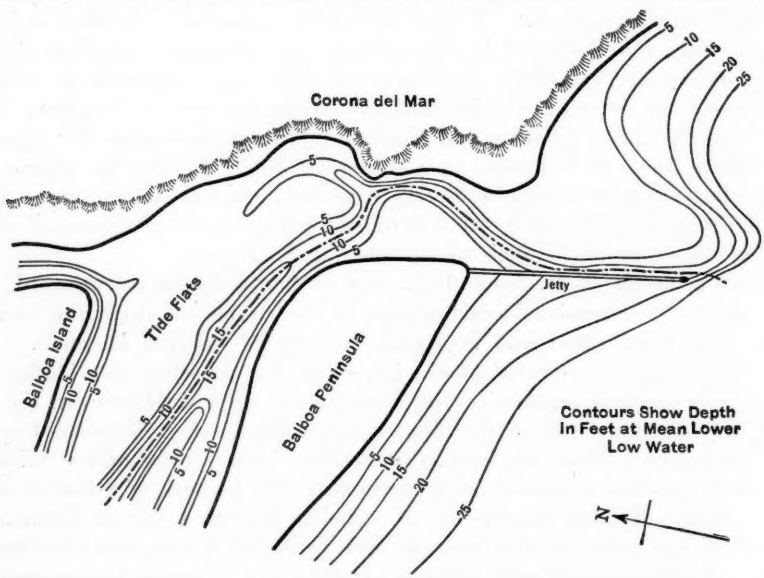

FIG. 10.

a sharp curve around the end of the sandspit, and there is no inner bar. There is a tradition that once during a heavy storm this channel was filled up and a new one opened at the east end of the lagoon. This new channel soon migrated back westward to its original and permanent location. A repetition

* "Relation of Depth to Curvature of Channels," *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 207.

of this action would now result in the destruction of much valuable beach property.

At Mugu Bay, in Ventura County, a still different condition prevails. Here, the indications are that the entrance channel was at one time at the east end of the lagoon at the foot of a steep hill. The detritus from this hill has filled up the channel and the entrance is now near the middle of the lagoon, with sandspits on both sides. This channel is constantly varying in character and position.

Improvements to the existing natural channels forming entrances to lagoons in this district, or the creation of artificial entrance channels to the closed lagoons for navigation purposes, should take into consideration the local geology. The natural channels show that the direction of the littoral drift is not the controlling factor in their location; also, that if the proper curvature of the inner end of the channel can be maintained, the formation of the inner bar and migration of the channel can be prevented. It then becomes a question of securing sufficient depth of water over the outer bar.

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PAPERS AND DISCUSSIONS

THE COMPRESSIBILITY OF SAND-MICA MIXTURES

Discussion*

By Messrs. R. D. N. Simham and William W. Rubey.

R. D. N. Simham,† Assoc. M. Am. Soc. C. E. (by letter).‡—From the few results of experiments that have been given by Mr. Gilboy, the writer has been able to deduce, in his own way, certain very important facts about soil behavior under compression, as follows.

General Relations for Volume of Mixture and Volume of Compression.—Let V_1 equal a volume of pure sand weighing W grammes and V_2, a volume of mixtures (loosely poured) of the same weight, W grammes; let V represent the compressed volume of mixtures under a specific load intensity, W_2 kg. per sq. cm. Then, if p is the percentage of mica by weight in the mixture, the volume of sand constituting the loose mixture of constant weight is given in each case by the relation,

$$V_s = \frac{V_1 (100 - p)}{100} \quad \ldots\ldots\ldots\ldots (1)$$

The volume of mica in the mixture will be given by,

$$V_m = \frac{V_1 \times p \times C}{100} \quad \ldots\ldots\ldots\ldots (2)$$

in which, C is a constant.

From Equations (1) and (2) the relation for the volume of mixture is given by,

$$V_2 = V_s + V_m = \frac{V_1 (100 - p)}{100} + \frac{V_1 \times p}{100} \times C \quad \ldots\ldots\ldots\ldots (3)$$

The writer computes the value of C to be approximately 7.4 for mica. V_1 is given as equal to 140 cu. cm., which is the volume of pure sand weighing 200 grammes. From Equations (1), (2), and (3), the values shown in Table 1 are derived, which may be compared with the values given in Fig. 3.§

* This discussion (of the paper by Glennon Gilboy, Jun. Am. Soc. C. E., published in February, 1928, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Town Planning Asst., Madras, India.

‡ Received by the Secretary, April 7, 1928.

§ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 561.

TABLE 1.—MICA CONTENT BY EQUATIONS (1), (2), AND (3).

Percentage of mica.	0	5	10	20	40	100
V_s	140	133	126	112	84	0
V_m	0	52	103	206	414	1030
V_2	140	185	229	318	498	1050

For the volumes of sand and mica content in the samples of the same weight but of different percentages, the following further general relations seem to hold good, and may be used as alternatives to Equations (1), (2), and (3):

$$V_s = W\ M_s\ (100 - p) \quad \ldots\ldots (4)$$

and,

$$V_m = W_1\ N_m\ p \quad \ldots\ldots (5)$$

in which, M_s and N_m represent volume coefficients for a unit weight (1 gramme) of mixture. W is the weight of samples tested, in grammes. From the results noted in Fig. 3, the values of these coefficients are found to be equal to 0.007 and 0.051 per gramme of mixture.

The volumes of mixtures with different percentages of mica will be given by,

$$V_2 = W\ M_s\ (100 - p) + W\ N_m\ p \quad \ldots\ldots (6)$$

The calculated volumes for mica, sand, and mixtures according to Equation (6) are noted in Table 2.

TABLE 2.—MICA CONTENT BY EQUATIONS (4) AND (6).

Percentage of mica.	0	5	10	20	40	100
V_s	140	133	126	112	84	0
V_m	0	51	102	204	408	1020
V_2	140	184	228	316	492	1020

As for the compressed volumes of mixtures (for a particular compression load, W_2), the following general relation seems to hold good:

$$V = W_2\ k\ V_s + W_2\ l\ V_m \quad \ldots\ldots (7)$$

in which, k represents the compressibility coefficient for sand and l the compressibility coefficient for mica. W_2 = load, in kilogrammes per square centimeter. $V_2 = V_s - V_m$ and varies with different percentages of mica. Equation (7) may also be expressed as:

$$V = \frac{V_1\ (100 - p)}{100}\ k\ W_2 + \frac{V_1\ p}{100}\ C\ l\ W_2 \quad \ldots\ldots (8)$$

In the cases of 200 grammes weight of samples tested by the author, with $V_1 = 140$ cu. cm. and $W_2 = 1$ kg. per sq. cm., the values of k and l are, respectively, equal to $0.99 \pm$ and $0.52 \pm$.

Relations for Constant Volume Mixtures.—From Equations (1) and (2) the following general relations for constant volume are deduced:

$$V_s = \frac{V_1\,(100 - p)}{100 + p\,(C - 1)} \qquad (9)$$

and,

$$V_m = \frac{V_1\,p\,C}{100 + p\,(C - 1)} \qquad (10)$$

The same value of $C = 7.4$ would apply; and $V_s - V_m = V_1 =$ constant. Substituting Equations (9) and (10), in Equation (7), the result is:

$$V = \frac{V_1\,(100 - p)}{100 + p\,(C - 1)}\,W_2\,k + \frac{V_1\,p\,C}{100 + p\,(C - 1)}\,W_2\,l \qquad (11)$$

General Relations for Determining Compression Under Load for Different Percentages of Mica.—For constant weight mixtures, $V_c = V_2 - V$. V_c represents the compression volume. Therefore,

$$V_c = V_1\left(1 - \frac{p}{100}\right)(1 - W_2\,k) + V_1\,\frac{p\,C}{100}\,(1 - W_2\,l) \qquad (12)$$

For constant volume mixtures,

$$V_c = \frac{V_1}{100 - p\,(C - 1)}\,((100 - p)\,(1 - W_2\,k) + p\,C\,(1 - W_2\,l)) \qquad (13)$$

For the sand mica samples the values of k, l, and C are, respectively, 0.99, 0.52, and 7.4. Correct values of these may be determined.

From the foregoing mathematical analysis the writer is led to believe that in any mixed soil the behavior under compression is one of combined action of the several ingredients, each behaving as if it were independent.

He believes that in the case of constant weight of samples of soil containing p_1, p_2, p_3, etc., percentages of different materials are mixed with sand of a percentage, p, then the loose volumes of mixtures will be given by,

$$V_2 = \frac{V_1\,p}{100} + \frac{V_1\,p_1}{100}\,C_1 + \frac{V_1\,p_2}{100} +, \text{etc} \qquad (14)$$

in which,

$$(p + p_1 + p_2 + \text{etc.}, = 100)$$

and, also, the final volume of soils when compressed by the load, W_2 kg. per sq. cm., will be given by,

$$V = \frac{W_2\,k\,V_1\,p}{100} + \frac{W_2\,l_1\,V_1\,p_1}{100}\,C_1 + \frac{W_2\,l_2\,V_1\,p_2}{100}\,C_2 +, \text{etc} \qquad (15)$$

Similar relations may also be deduced for soils when V_2 is constant. The values of the constants, k, l_1, l_2, etc., and C_1, C_2, etc., for different materials constituting the soils mixture, should be carefully determined by experiment.

The results furnished by the author are indeed very enlightening in so far as the question of compressibility of soils under peculiar conditions is concerned; but similar attempts should also be made to determine constants of permeability, slipping or sliding, elasticity, and friction. As soils sink down under most of these conditions also, constants for soils for each condition should be determined so that the behavior of any mixed soil may be correctly computed and estimated, on more or less the same lines as herein suggested.

WILLIAM W. RUBEY,* ESQ. (by letter).†—This paper, dealing with the effect of differences of composition on the deformation of mixtures that are similar to sedimentary rocks, is of interest to geologists as well as to engineers. The compactibility of clays and related fine-grained rocks has been given some study by geologists,‡ but never under such controlled conditions as those adopted by Mr. Gilboy. Consequently, his experiments throw new light on some important phases of rock deformation.

The petrographic microscope has shown that platy micaceous crystals§ are a very common constituent of clays, shales, and mudstones, and his experiments furnish confirmatory evidence that these platy crystals may account for some of the peculiar properties of these types of rocks. However, the microscope has also shown that the particles in many of these rocks are very small; and physical investigations have indicated that the proportion of these small particles is an important characteristic of soils and clays.‖ These two interpretations are not mutually exclusive. It is possible that both the shape and the size of the constituent grains may influence the behavior of a rock under compression, for the surface area per unit volume increases with both smallness and flatness of the grains. In other words, some of the properties of clays and soils may be a function of their total or specific internal surface.¶

With these earlier investigations in mind, Mr. Gilboy's conclusions** that "the characteristic behavior of fine-grained sediments, such as silts and clays, is principally ascribable to the presence of flat grains" and that "the compressibility of typical fine-grained sediments is not ascribable to the small size of the grains nor to the presence of colloidal particles" are not completely convincing. His samples of sand and mica were sized with sieves and, therefore, although in maximum or intermediate diameters the two kinds of grains are the same size, in volumes or true sizes the spherical quartz grains are many times larger than the thin plates of mica. If the two kinds of

* Associate Geologist, U. S. Geological Survey, Washington, D. C. Published with the permission of the Director, U. S. Geological Survey.

† Received by the Secretary, April 17, 1928.

‡ King, F. H., "Principles and Conditions of the Movements of Ground-Waters," U. S. Geological Survey, 19th Annual Rept., Pt. 2, pp. 77–85, 1899; Sorby, H. C., "On the Application of Quantitative Methods to the Study of the Structure and History of Rocks," *Quarterly Journal*, Geological Soc., London, Vol. 64, pp. 227-231, 1908; and Hedberg, H. D., "The Effect of Gravitational Compaction on the Structure of Sedimentary Rocks," *Bulletin*, Am. Assoc. Petroleum Geologists, Vol. 10, pp. 1035–1072, 1926.

§ Somers, R. E., "Microscopic Study of Clays," *Bulletin 708*, U. S. Geological Survey, pp. 296–298, 1922; Wherry, E. T., "Bentonite as a One-Dimensional Colloid," *American Mineralogist*, Vol. 9, pp. 120–123, 1925; Freundlich, H., "Soils with Non-Spherical Particles," *Second Colloid Symposium Monograph*, pp. 46–56, 1925; and Ross, C. S., and Shannon, E. V., "The Minerals of Bentonite and Related Clays and Their Physical Properties," *Journal*, Am. Ceramic Soc., Vol. 9, pp. 77–96, 1926.

‖ Ashley, H. A., "The Colloid Matter of Clay and Its Measurement," *Bulletin 388*, U. S. Geological Survey, 1909; Hardy, F., "The Physical Significance of the Shrinkage Coefficient of Clays and Soils," *Journal of Agricultural Science*, Vol. 13, pp. 243–264, 1923; Russell, J. C., and Burr, W. W., "Studies on the Moisture Equivalent of Soils," *Soil Science*, Vol. 19, pp. 251–266, 1925; Wintermeyer, A. M., "Adaptation of Atterberg Plasticity Tests for Subgrade Soils," *Public Roads*, Vol. 7, pp. 119–122, 1926; and Anderson, M. S., and Mattson, Sante, "Properties of Colloidal Soil Material," *Bulletin 1452*, U. S. Dept. of Agriculture, 1926.

¶ Mitscherlich, E. A., and Floess, R., "Ein Beitrag zur Bestimmung der Hygroskopizität und zur Bewertung der physikalischen Bodenanalyse," *Internationale Mitteilungen für Bodenkunde*, Bd. 1, pp. 463–480, 1911; Baker, H. A., "On the Investigation of the Mechanical Composition of Loose Arenaceous Sediments by the Method of Elutriation," etc., *Geological Magazine*, Vol. 57, pp. 363–370, 1920; and Sauramo, Matti, "Studies on the Quarternary Varve Sediments in Southern Finland," *Bull. 60*, de la Commission Géologique de Finlande, pp. 17–19, 1923.

** *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, pp. 555, 568.

grains had been sized by any of the "sedimentation" methods of mechanical analysis instead of by sieving, the mica grains would probably have appeared much smaller than the quartz grains.

Mr. Gilboy's data on the dimensions of the grains and on the effect of varying proportions of sand and mica on the void ratios and compactibilities of the mixtures afford an opportunity to test the hypothesis that the chief factor is the internal surface area of a rock instead of either the size or shape alone of its constituent particles. The sand he used was rounded, had an average diameter, D, of 0.5 mm., and a specific gravity, G, of 2.66. The mica passed the same sieves, was about 0.02 mm. thick, T, and had a specific gravity of 2.85. Therefore:

	Average Quartz Sphere.	Average Mica Disk.
Surface area......	$\pi D^2 = 0.785$ sq. mm.	$\frac{2\pi D^2}{4} + \pi D T = 0.424$ sq. mm.
Relative weight...	$\frac{\pi D^3 G}{6} = 0.174$	$\frac{\pi D^2 T G}{4} = 0.0112$

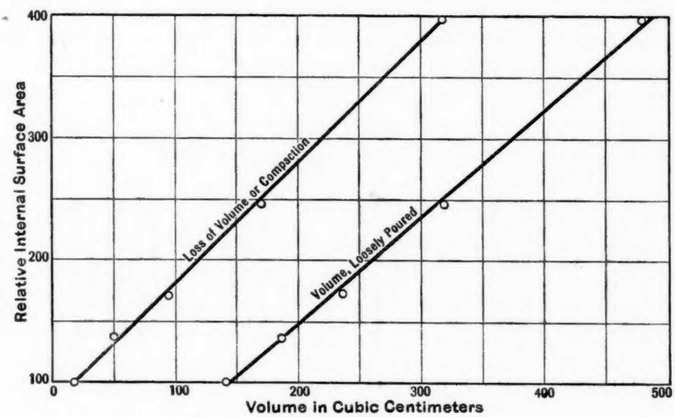

FIG. 5.

That is, the average sand grain had less than twice the surface area of the average mica flake, but was about 16 times as heavy. Per unit weight the mica had about 8.4 times as much surface area as the sand. Using this factor of 8.4 for the mica, the relative internal surface areas of the different mixtures can be computed from the weight proportions of each constituent (Column (2), Table 3). It is of considerable interest to note that the volumes of the loosely poured uncompacted mixtures (Column (3)) show an approximately linear relation to these surface areas (Fig. 5).

Because of the friction on the sides of the graduate, Mr. Gilboy warned readers against using the compacted volumes of the mixtures that were shown in Fig. 3.* However, the true relative compacted volumes can be estimated from the void ratios of these mixtures at a pressure of 10 kg. per sq. cm. as shown in Fig. 2† (see Table 3, Column (4)). To convert these void

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 561.
† *Loc. cit.*, p. 560.

ratios into relative volumes, one must know the average specific gravity of the mineral particles in the different mixtures, and this can be computed from the formula,

$$\text{Average Specific Gravity of Mineral Particles} = \frac{100}{\dfrac{\text{Percentage of sand}}{2.66} + \dfrac{\text{Percentage of mica}}{2.85}}$$

The values, thus computed, are shown in Table 3, Column (5). Then the volumes of 200-gramme samples of the different mixtures, at pressures of 10 kg. per sq. cm., would be,

$$V = \frac{200\ (1 + \text{void ratio})}{\text{Average specific gravity of minerals}}$$

as shown in Table 3, Column (6); and the loss of volume, or the compactibility of a 200-gramme sample, when subjected to a pressure of 10 kg. per sq. cm., would be (at least, approximately) the difference between the volume when loosely poured and the volume when compacted (Table 3, Column (7)). These differences or amounts of compaction are essentially a linear function of the relative surface areas of the mixtures (see Fig. 5).

TABLE 3.

Percentage mica in sand-mica mixture. (1)	Relative internal surface area of mixture. (2)	Volume, in cubic centimeters, of 200 grammes of the mixture, loosely poured. (3)	Void ratio of mixture at pressure of 10 kg. per sq. cm. (4)	Average specific gravity of the mineral particles. (5)	Volume, in cubic centimeters, of 200 grammes of the mixture, compacted by a pressure of 10 kg. per sq. cm. (6)	Loss of volume on compacting (Column (3) − Column (6)). (7)
0	100	140	0.63	2.66	123	17
5	137	185	0.81	2.67	136	49
10	174	237	0.92	2.68	143	94
20	248	320	1.02	2.70	150	170
40	396	480	1.22	2.73	163	317

These two apparently linear relationships seem to indicate that the volumes of these mixtures when uncompacted and their losses of volume when subjected to equal increments of pressure, depend on their internal surface areas rather than on either the shapes or the sizes of the particles alone. This suggests that compacting takes place, not by the bending of thin plates, nor by the nestling together through the rolling about of small spheres, but possibly by the squeezing out of cushion-like films that cover the surfaces of all particles. At least, it seems that an explanation of the apparent importance of internal surface areas might profitably be made a goal of future experiments on compacting.

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PAPERS AND DISCUSSIONS

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SILTING OF THE LAKE AT AUSTIN, TEXAS

Discussion*

By Messrs. H. F. Robinson, Julian Montgomery, Kirk Bryan, and J. C. Stevens.

H. F. Robinson,† Esq. (by letter).‡—In connection with the observations of the author, the status of the silting problem at the Zuni Reservoir in New Mexico is of interest.§ The conditions affecting the two cases are quite dissimilar. At Austin, there is a long, narrow reservoir in a perennial stream and, at Zuni, a reservoir almost circular in shape is formed in the bed of an ephemeral stream which is supplied by a very uncertain rainfall and run-off. In both cases large quantities of silt have been brought in and deposited; at Austin, 95.39% of the storage capacity in 13 years, and at Zuni, at the 990-ft. contour, 94.89% in 22 years.

The showing of such immense quantities of silt being brought into reservoirs, under dissimilar conditions, when constructed in stream channels, especially in this western country where erosion is increasing from year to year, is not only astounding, but alarming.

From his observations on the Zuni Reservoir, the writer cannot quite agree with the theory of silt deposit advanced by Dean Taylor. Ordinarily, the amount of silt deposited in a reservoir depends on (*a*) the quantity of water coming into, or passing through, the reservoir; and, (*b*) the amount of silt carried by the water. The latter is a variable figure depending on many things, including the character of the drainage area. Under the conditions existing at both reservoirs there is coming into, or through, the reservoir a certain quantity of water each year, which water will carry an unknown but variable amount of silt. It would seem that irrespective of the size of the reservoir (within reasonable limits), with a given quantity of water going into or

* Discussion of the paper by T. U. Taylor, M. Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Superv. Engr., U. S. Indian Irrig. Service, Albuquerque, N. Mex.

‡ Received by the Secretary, April 3, 1928.

§ See, also, *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 868.

through it, there would be deposited a similar volume of silt; and as the area of the reservoir was reduced, the deposit of silt would be greater in depth in each period.

Of course, a given quantity of water coming into a large, empty reservoir and remaining there would deposit a greater amount of silt than would the same quantity of water flowing into a reservoir already full, or partly full, and passing over the spillway or dam, carrying a portion of its silt. However, this would be only one minor factor affecting the result.

The "Theory of Silting", as stated* by the author, would seem to take none of these factors into consideration, but rather to assume that, knowing the capacity of the reservoir before and after silting and the period elapsing, the rate of deposit is wholly a function of the capacity of the reservoir.

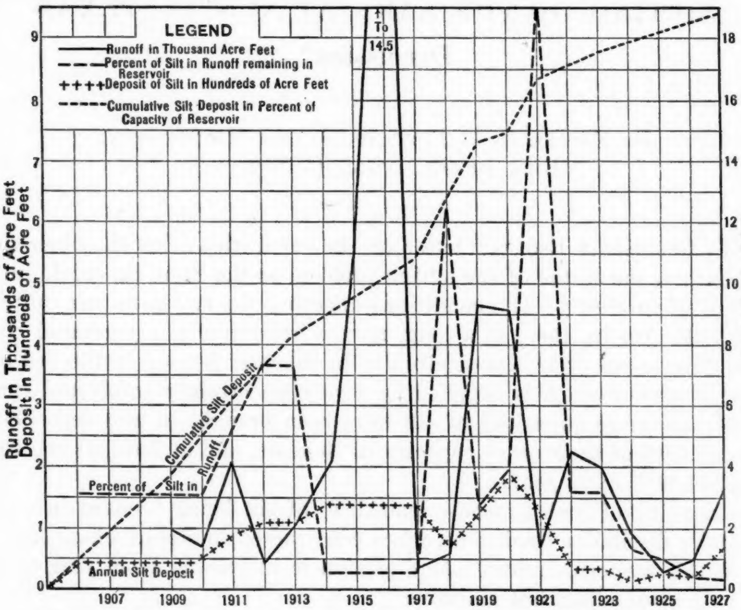

FIG. 7.—SILT PROBLEM OF THE ZUNI RESERVOIR AT THE 990-FOOT CONTOUR.

The records of the Zuni Reservoir (Table 4) seem to indicate an irregular deposit which has no reference to the capacity of the reservoir and only indirectly as to (*a*) the quantity of water, with (*b*) the silt content being almost undetermined. The curve of silt deposited annually will be practically a straight line (see Fig. 7†) rather than a curve as shown in the author's Fig. 5.‡

It is perhaps advisable to give the conditions that have been found at Zuni. After the failure of 1909 the weir was built to Elevation 991, but since then a permanent weir of concrete has been constructed to Elevation 997 and stop-planks can be used to raise it to the original 1 000-ft. elevation.

* *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 576.

† See, also, *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 871, Fig. 1.

‡ *Proceedings*, Am. Soc. C. E., February, 1928, Papers and Discussions, p. 577.

TABLE 4.—SILT DEPOSITS IN ZUNI RESERVOIR.

(Elevation 997, top of weir = Gauge 44.8
" 998.3 " " stop-logs = " 46.1
" 1 000 " " " = " 47.8
Dam not yet high enough to carry water above 1 000.)

Year.	Total silt deposit cumulative, in acre-feet.	Length of period, in years.	Silt deposit for period, in acre-feet.	Total run-off for the period, in acre-feet.	Percentage of silt in run-off remaining in reservoir.	At Contour 990.			Present Maximum at Contour 1 000.			Dam to be Raised to Contour 1 010.		
						Capacity.	Percentage remaining of original capacity.	Average lost capacity per year in percentage of original.	Capacity.	Percentage remaining of original capacity.	Average lost capacity per year in percentage of original.	Capacity.	Percentage remaining of original capacity.	Average lost capacity per year in percentage of original.
1906	0	....	.....	38 000	.....	10 230	....	....	15 811	....	....	20 389	....	...
1907	......	....	.....	....	.....	......	....	4.4	......	....	2.9	......	....	2.2
1908	......	....	.....	18 140*	.....	......	....	4.4	......	....	2.9	......	....	2.2
1909	......	....	.....	....	.....	......	....	4.4	......	....	2.9	......	....	2.2
1910	1 804	4	1 804	24 630†	3.2	8 409	82.3	8.8	14 007	88.4	5.7	18 585	91.1	4.4
1911	3 144	1½	1 340	....	5.4	7 069	69.2	8.8	12 667	80.0	5.6	17 245	84.5	4.4
1912	......	....	.....	4 030	.....	......	....	5.2	......	....	3.3	......	....	2.5
1913	4 200	2	1 056	10 330	7.3	6 013	58.8	5.2	11 611	73.4	3.3	16 189	79.4	2.5
1914	......	....	.....	21 150	.....	......	....	3.2	......	....	1.9	......	....	1.6
1915	......	....	.....	54 811	.....	......	....	3.2	......	....	1.9	......	....	1.6
1916	......	....	.....	146 588	.....	......	....	3.2	......	....	1.9	......	....	1.6
1917	5 507	4	1 307	3 777	0.58	4 706	46.0	3.2	10 304	65.8	1.9	14 882	72.9	1.6
1918	6 185	1	678	5 323	12.5	4 028	39.3	6.7	9 626	60.8	5.0	14 204	69.6	3.3
1919	7 433	1	1 248	46 473	2.66	2 801	27.3	12.0	8 292	53.0	7.8	12 956	63.3	6.1
1920	9 296	1	1 863	45 711	4.0	2 503	24.5	2.8	7 308	41.3	11.7	11 120	54.5	9.0
1921	10 589	1	1 291	6 770	19.0	1 666	16.28	8.22	6 114	38.2	3.1	9 829	48.2	6.3
1922	......	....	.....	22 723	.....	......	....	2.1	......	....	1.9	......	....	1.5
1923	11 271	2	684	20 375	2.96	1 248	12.1	2.1	5 430	34.3	1.9	9 242	45.3	1.5
1924	11 388	1	117	8 865	1.30	1 074	10.5	1.6	5 313	33.6	0.7	9 125	44.4	0.9
1925	11 653	1	265	2 645	1.00	925	9.04	1.5	5 048	31.9	1.7	8 860	43.4	1.0
1926	11 839	1	186	4 981	0.38	728	7.11	1.9	4 862	30.7	1.2	8 674	42.5	0.9
1927	11 940	1	606	16 017	0.38	541	5.29	1.8	4 256	26.9	3.8	8 605	42.2	0.3

* January, 1908, to June, 1910.
† June, 1910, to January 1, 1912.

Silt Deposit.—The quantity of silt brought into the reservoir has been very great. Records are now complete to January 1, 1928, a period of approximately 22 years (Table 4). With the water surface in the reservoir at Contour 990, there is only 5.29% of the capacity left. The amount of silt brought into the reservoir, however, has been greater than this would indicate, the excess being deposited between the 990 and 1 000-ft. contours. The total capacity at Contour 990 was 10 230 acre-ft. The total silt deposit in the reservoir January 1, 1928, was 11 940 acre-ft. The area of the reservoir at the 990-ft. contour is 512 acres and at the 1 000-ft. contour, 620 acres. The capacity remaining at the 1 000-ft. contour is 26.9% of the original capacity of 15 811 acre-ft. at that height.

The first attempt to determine the amount of silt coming in was in June, 1910, when soundings were taken from a boat, and at intervals thereafter soundings were made through the ice, usually in January. There are only a few weeks in any winter when the ice is thick enough for this purpose and in several years it was impossible at any time in the winter to get out on the ice. This was the case for the several years of no record (Table 4).

In making these soundings the same base line was used each year and the surface of the reservoir was divided into squares 300 ft. on the side. This assured that the soundings for each year were made at practically the same spot and the records, therefore, are unusually close. From these soundings contour plats were made from which the amount of silt deposited was computed. Table 4 shows in detail the silt deposited by periods and the total run-off entering the reservoir for the same periods. The capacity of the reservoir at Contours 990, 1 000, and 1 010 are shown, with the percentages remaining of the original capacity at each of the periods under consideration.

The reservoir may be considered defined by the 990-ft. contour. Many of the data have been recorded in Fig. 7. In order to compare the various elements, the run-off is expressed in thousands of acre-feet, the deposit in hundreds of acre-feet, and the percentage of silt in the run-off which remained in the reservoir. This may assist in an understanding of conditions and of the original statement* that "the percentage of silt carried by the water is in a certain inverse ratio to the magnitude of the run-off, within, as yet, undetermined points."

Two important considerations that were not stated modify that conclusion. For one thing the statement is true only so long as the regimen of the run-off area remains undisturbed. Again, this statement and the data given in Table 4 refer to the percentage of silt in the run-off. This latter is not entirely true, for what is shown is the percentage of silt in the run-off which remains in the reservoir. A certain amount will remain in suspension and pass over the spillway and the part dropped in the reservoir will depend on many conditions, such as the state of the reservoir when a given run-off occurs; whether empty or full; the character of the silt, etc. Hence, the writer is not as sure as formerly that there is any intimate connection between the amount of silt carried by a particular flood and its magnitude. It should also be remembered that a part of the silt does not remain in the reservoir permanently because

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 888.

some is carried out with the irrigation water. It has been estimated that at least 50 acre-ft. goes out in that way annually at the Zuni Reservoir. However, the record is very clear that the greater the run-off the less will be the percentage of silt carried by the water.

The theory of silt deposit of Dean Taylor probably fits the conditions and results at the Austin Dam, but will not fit those at the Zuni Reservoir. The cumulative curve of the silt deposit on the Zuni chart (Fig. 7) lies fairly close to a straight line from 1906 to 1923. At this time changes in the drainage area took place in the nature of silt retardation work, and from 1923 to 1928 the silt curve again becomes, with slowly changing conditions, almost a straight line.

Prevention of Erosion.—There being no practicable method to remove silt once it reaches and is deposited in a large reservoir, it would appear that the proper procedure is to prevent, as far as possible, further silt being carried into it. During the early years of the Zuni Reservoir it appeared that most of the silt came from the erosion of the stream above and particularly from the branches coming in from the south, flowing through what is known as Horsehead Canyon, rather than the general erosion of the entire drainage area. This was undoubtedly caused by overgrazing, removing the ground cover, and allowing the run-off to reach the stream in a shorter space of time and in greatly augmented quantities.

During these early years very little erosion took place in the tributaries from the north. About 1922 it was found that heavy erosion had commenced on these tributaries, particularly on the Nutria. At the same time examinations led to the belief that the erosion on the southern tributaries had about reached an equilibrium and the danger of further heavy deposits from that direction was practically removed.

There are two phases of erosion in the drainage area of streams. One is the lowering of the channel bed and the retrogression of grades through the fine alluvial deposit of the valleys, which continues until bed-rock or hardpan is reached, the channel widening at the same time in proportion to its increased depth. The other, which is really a part of the same action, occurs where the stream carrying the flood has a sharp bend in which the water striking the outer bank of the curve, breaks off large masses and the eroded material is carried down the stream. Both these conditions are usually found in the same stream.

It was planned to check this erosion under the first mentioned condition by constructing rock and brush checks which are placed not only in the bottom, but are carried up the banks to a sufficient height to prevent bank erosion. It was planned to place these checks close enough together to form a new grade so that the velocity of the water will not exceed what it originally was, and the current will not erode the material (Fig. 8).

In the other case, masses of brush and rock were placed at the critical points along the side to prevent erosion. The protection at sharp bends as illustrated in Fig. 9 will not always work, for the simple reason that if the bottom of the arroyo is still in soft material the current will erode under the protection work; but in such cases it is only necessary (*a*) to excavate down

to harder material; (*b*) to build a protection mass so it will settle with the scouring; or (*c*) to use this side protection in connection with the checks.

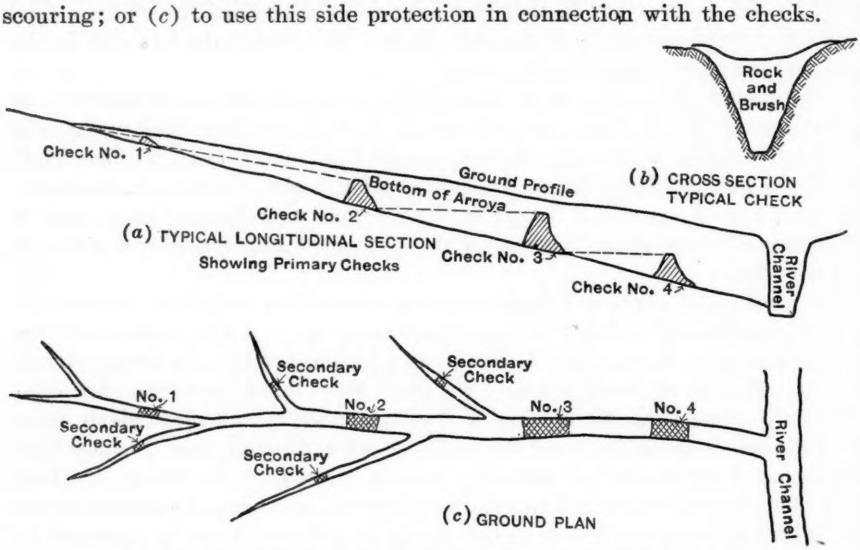

FIG. 8.—DIAGRAMMATIC PLAN OF CHECKING EROSION IN CHANNELS AS APPLIED TO THE ZUNI PROJECT.

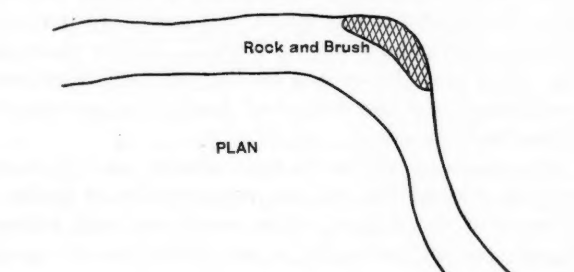

FIG. 9.—CHECKING EROSION ON SHARP TURNS IN STREAM.

The erosion on the north side of the water-shed assumed alarming proportions only in 1921 and 1922, and in 1923 a small sum of money was expended in the protective work outlined. A limited amount of this work has been continued each year since, at the most critical places.

That this has been effective is shown by the fact that the reservoir capacity curve (or what is the same thing, the curve showing the silt deposit (Fig. 10)) is suddenly flattened from 1922 onward. This indicates clearly, unless several consecutive coincidences have occurred, that the protective checks and jetties are doing their work.

An extension of the control work is planned from year to year as money is available and at points indicated as necessary by further erosion.

Processes of Erosion.—As to the general problem of erosion in the Southwest, an office "Watershed Handbook" prepared by the U. S. Forest Service, Southwestern District, contains some valuable comments which may well be repeated in substance as bearing directly on the large question.

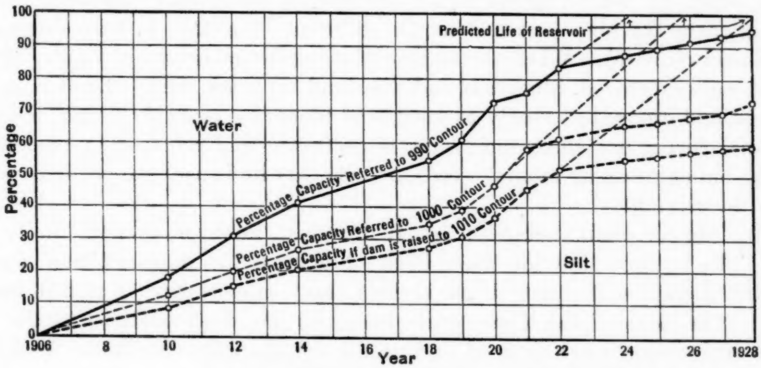

FIG. 10.—SILTING OF ZUNI RESERVOIR. CAPACITIES IN PERCENTAGES OF VOLUMES BELOW GIVEN CONTOURS.

The distinction between normal and abnormal erosion is a frequent cause of misunderstanding. A certain kind and rate of erosion, of course, exist naturally in every place, but are seldom the same in any two places.

For any locality they are determined by the interplay of two forces:

1.—Resistance.—Vegetative obstruction, growth, or healing power of vegetation, kind and arrangement of soil, rocks, and débris.

2.—Disintegration.—Kind, amount, and distribution of rainfall, water flow, freezing and thawing, wind, etc.

Whenever, through a long period of time (say, a generation) these two forces approximately balance, and no material change in the configuration of the country occurs, the country may be said to be in equilibrium, and the state of erosion may be called normal. Whenever, through a similar period of time, a material change occurs (such as the disappearance of creek bottoms or gullying of slopes) the state of erosion may be called abnormal. Abnormal erosion occurs locally in the Southwest while the land is still in its virgin condition, but it was exceptional, both as to time and place, because the natural forces which upset the equilibrium (cloudbursts, for instance), were local and temporary. The natural forces for restoring equilibrium (vegetation, for instance), usually had time to "get in" their work before the upsetting process was repeated.

The artificial forces which are now upsetting equilibrium (over-grazing, for instance), are usually not local and temporary, but widespread and of long standing. Hence, the fact that abnormal erosion, once exceptional, is now general. Once self-healing, it is now cumulative. This argument does not assume that there are more or worse cloudbursts now than before the country was settled. It does assume, however, that the same number and kind of cloudbursts have more effect. For practical purposes, no distinction need be made between normal and abnormal. Any erosion that is doing damage and that can be controlled, should be controlled. There is no danger of the effort being so successful as to interfere with normal or beneficial erosion.

Why should settlement upset equilibrium on a wholesale scale in the Southwest, when in other regions it has not usually done so? Because, for one thing, there the plant successions are different (probably in turn due to the semi-arid climate). When the prairie grass and timber groves of the Middle West were destroyed, bluegrass and plowland replaced them. Land regularly plowed and cropped is nearly as good, and bluegrass probably better, than the original cover; but when the grama is destroyed and replaced by snakeweed, or the bunch-grass by annual weeds, or side-oats by brush, the cover is poorer than the original.

An inferior plant succession is not universal, however, even in the Southwest. In regions like the East Pecos, bluegrass has replaced bunch-grass. On the Jemez, *valles* sedge and various short grasses have replaced bunch-grass. On parts of the Tonto curly mesquite has replaced the gramas. All these replacements are as good or better cover than the original. As a rule, however, the replacements have been inferior.

Restoration of equilibrium eventually occurs naturally, even where upsetting forces continue at work. The best example is in the region of Santa Fé, where the lower country around the Spanish settlements has been over-grazed for three centuries. The watercourses have simply eroded until they exposed enough rock and cobbles to furnish a mechanical resistance equal to the original vegetative resistance. Thus stabilized, the watercourses are proceeding to re-vegetate with alder, juniper, rabbit-brush, or whatever can grow in the rocks. The new equilibrium, however, is a relatively unproductive one from the economic standpoint.

Julian Montgomery,* M. Am. Soc. C. E. (by letter).†—Engineers who are concerned with the design and development of storage reservoirs in the southwest section of the United States should greatly appreciate this valuable contribution.

The geology of the drainage area, together with the relation between the size of that area and the capacity of the storage reservoir, are factors that designing engineers must constantly keep before them.

Familiarity with the hydrology of the Southwest reveals that the run-off of the water-sheds of the streams in that section carry considerable silt. It is not unusual for engineers engaged in the design of storage reservoirs to allow from 0.3 to 0.5 acre-ft. of siltage per year per square mile of drainage area.

* Cons. Engr. (Montgomery & Ward), Wichita Falls, Tex.

† Received by the Secretary, May 14, 1928.

The relation of the drainage area above Lake Austin, in square miles, to the storage capacity of old Lake Austin, in acre-feet, was approximately 1 to 1.6.

Lake Wichita, the water supply for Wichita Falls, Tex., was built on Holliday Creek about 1900. It has a drainage area of 135 sq. miles and a storage capacity of 14 500 acre-ft. The geology of the water-shed is largely Permian. There is considerable silt in the run-off. Yet flood waters pass over the spillway with sufficient regularity to prevent any particular accumulation of silt. Cross-sections taken in 1925 under the supervision of Dean Taylor show very little siltage in the twenty-five years. The cross-sections also show that scouring has taken place, in general, along the path of the current. At the breaks and curves along the shore line silt has been deposited. Probably the average net deposit over the entire reservoir area would be less than 1 ft. for the life of Lake Wichita. The relation of the drainage area, in square miles, to the capacity of Lake Wichita, in acre-feet, is 1 to 110, approximately.

Lake Kemp, located on the Big Wichita River, approximately 50 miles west of Wichita Falls, has a drainage area of 2 000 sq. miles and a storage capacity of approximately 500 000 acre-ft., a ratio of 1 to 250.

KIRK BRYAN,* ESQ. (by letter).†—The quantitative data presented by Professor Taylor on the deposition of silt in Austin Reservoir add valuable information on the processes of erosion and deposition characteristic of streams in the arid Southwest. These streams may be divided into three classes:‡ (1) Perennial, or those that flow nearly all the time; (2) intermittent, or those that flow only part of the time but more than a month a year; and (3) ephemeral, or those that flow only after rains and less than a month a year.

The perennial streams carry some silt at low-water stages and may be particularly active during the closing part of the flood-water period. Ephemeral and, to a less extent, intermittent streams carry silt only during floods.

Erosion and deposition of silt by such streams are subject to changes and fluctuations quite distinct from the same processes in perennial streams. Thus, the Lower Gila in Arizona and the Rio Grande in New Mexico show a tendency to aggrade their beds, whereas their ephemeral tributaries are actively degrading their channels. This accelerated erosion, or arroyo cutting,§ was begun within the memory of men now living. It has vastly increased the total quantity of silt carried by the ephemeral streams. This increased quantity of silt menaces reservoirs constructed to store flood waters. Simi-

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* Asst. Prof. of Physiography, Harvard Univ., Cambridge, Mass.

† Received by the Secretary, May 26, 1928. This material formed part of a report to the Middle Rio Grande Conservancy District, September, 1927, on "Erosion and Control of Silt on the Rio Puerco, New Mexico," by Kirk Bryan and George M. Post, published by permission of J. L. Burkholder, M. Am. Soc. C. E., Chief Engineer.

‡ Note the excellent definitions of O. E. Meinzer, "Outlines of Ground Water Hydrology," U. S. Geological Survey *Water Supply Paper No. 494*, pp. 57–59, 1923.

§ Bryan, Kirk, "Date of Channel Trenching (Arroyo Cutting) in the Arid Southwest," *Science*, n.s., Vol. 62, pp. 338–344, 1925; "Channel Erosion of the Rio Salado, Socorro County, New Mexico", U. S. Geological Survey, *Bulletin 790*, pp. 17–19, 1927; "Historic Evidence of Changes in the Channel of Rio Puerco, a Tributary of the Rio Grande in New Mexico," *Journal of Geology*, Vol. 36, pp. 265–282, 1928; also, Swift, T. T., "Date of Channel Trenching in the Southwest", *Science*, n.s., Vol. 63, pp. 70–71, 1926; and Wynn, Fred, "The West Fork of Gila River", *Science*, n.s., Vol. 64, pp. 16–17, 1926.

larly, in the valleys of the major streams, the abnormal supply of sediment chokes the channels of perennial streams, and induces a number of consequences most of which are troublesome to the works of Man.

The conditions of erosion and sedimentation on the water-sheds of the ephemeral streams have been the subject of much speculation and study with a view of determining the cause of the accelerated erosion or arroyo cutting. A theoretical analysis of the conditions indicates that, because of the increased growth of vegetation, years of large precipitation will be times of minimum erosion by running water and, consequently, times of minimum silt transfer to the lower reaches of the stream. In these periods of larger rainfall, greater quantities of water will flow down the hill slopes through the minor channels, but the erosive power of this water will be much reduced because of the increased vegetative cover. Not only does this principle seem to apply to the short periods of one or two years which frequently occur at present and are recorded in existing weather records, but many claim that it will also apply to longer periods. Thus, in analyzing the effect of possible fluctuations in climate in past times, this principle has been used. Periods of erosion are thought to be the result of arid climate and periods of alluviation and filling of stream channels are attributed to less arid climate. In regions of greater rainfall and on perennial streams it seems likely that the principle will not hold and the results of a change in climate may be exactly reversed.

Although this principle has been advocated by eminent men and seems sound so far as a mere analysis may go, no definite evidence has heretofore been brought forward. The experience in the operation of the Zuñi Reservoir, however, gives qualitative evidence that this principle is true.

Zuñi Reservoir on the ephemeral Zuñi River in McKinley County, New Mexico, acts as a trap to retain the sand and mud carried by the stream. The area of the water-shed above the reservoir is said to be about 650 sq. miles. Data on the river, its reservoir, and the silt problem have been published by H. F. Robinson,* Supervising Engineer, U. S. Indian Irrigation Service, who has also supplied additional information used herewith.

The silt produced by the Zuñi water-shed is variable in absolute quantity and in relation to the water entering the reservoir. Twelve determinations of the quantity of silt deposited in the reservoir have been made, of which seven are for single years, and five for periods ranging from 1½ years to 3½ years. If the quantity of silt for these periods be divided by the mean annual discharge from the reservoir for the same period, as shown by Mr. Robinson in Table 4,† there is obtained an average figure for the silt in percentage by volume carried in the muddy water.

If these mean discharges are plotted as ordinates and the percentages of silt as abscissas, as shown by Mr. Robinson in Fig. 8,‡ the points representing the periods down to 1924 show a characteristic relation between the run-off from the water-shed and the silt content of the water. The points representing

* "Silt Problem of the Zuñi Reservoir," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), pp. 868-878, and discussion, pp. 879-893.

† See p. 1941.

‡ See p. 1944.

1923, 1924, 1925, 1926, and 1927, are affected in their position by the silt-control works begun in 1923. The points representing the earlier periods fall fairly close to a smooth line. The discrepancies may be due to errors in determination or to some natural condition. The period ending June, 1910, covers 3½ years, and the quantity of silt should be more accurately determined than in the period of 2½ years ending in January, 1912, represented by the near-by point; but the discharges from the reservoir for 1906 to 1908, inclusive, were not precisely determined and the estimate may be sufficiently in error to cause a misplacement of the point for the period, 1910. The points representing 1920 and 1921 appear to be in error as a result of miscalculation of the quantity of silt shown by the survey of 1920.

The building of structures to prevent excessive erosion was begun in 1923. On the next measurement of silt in January, 1924, the quantity deposited in Zuñi Reservoir was relatively small and the proportion of silt in the inflow fell to 2.96 per cent. In succeeding years, in spite of low run-off for the water-shed, the percentage of silt continued to decrease. As may be seen in Mr. Robinson's Fig. 8, these points are grouped together at the lower left-hand corner of the diagram. The fact that on the introduction of the artificial factor of silt control the curve based on previous data no longer applies, gives support to its validity as representing a true relation. The conclusion cannot be escaped that the greater the run-off, the less the silt in the water. The absolute quantity of silt either decreases or increases slightly with greater discharges, but so slightly that it is almost true that the more water, the less silt. Thus, in rainy years when more rain falls on the land, when the floods in the arroyos are larger and more prolonged, the percentage of silt in the main stream is lower, and the absolute quantity carried is but little, if at all, increased.

The same type of analysis was applied to the silt determinations on the Rio Grande, at San Marcial, made by the late W. W. Follett,* M. Am. Soc. C. E., during the years 1897-1912. As might be expected there is no similarity in the relation of discharge and silt content, either in the figures for full years or any of the natural seasons of the year. Analysis of the data on silt at and near San Carlos, on the Gila River, as compiled by Hughes† gave a somewhat similar type of curve with numerous erratic years. The Gila River at this locality is intermittent or even ephemeral in its major discharge, as the low-water flow is inconsiderable. It should furnish a similar curve and the erratic points may be ascribed to errors in the estimates. Data on other streams by which a further confirmation of the relationship shown for the Zuñi River between silt and discharge may be established, are lacking.

Obviously, if the water-shed of the Zuñi River were bare ground, the decrease of silt with run-off would be impossible. Some agent dependent on rainfall must intervene to prevent erosion at times of large run-off. This agent is the vegetative cover which is much reduced in effectiveness in dry years. Hence, the more rain, the more grass, and, consequently, the less erosion and the less silt carried in the streams.

* Follett, W. W., "Silt in the Rio Grande"; Commission on Equitable Distribution of Waters from the Rio Grande, Special Rept. to the Secy. of State, State Dept., 1913.

† Hughes, D. E., "Amount of Silt That Would Be Deposited in a Reservoir at San Carlos," etc., H. R. Doc. 791, 63d Cong., 2d Sess., pp. 117–140, 1914.

Students of forestry and grazing have long contended that artificial reductions of the vegetation by overgrazing promotes erosion and increases the silt content of streams. Many writers, following this line of reasoning, have held that the destruction of grass cover by overgrazing is the cause which induced the cutting of existing arroyos. On the other hand, others hold that a change in climate by which the rainfall is reduced, will produce the same result.

The data presented in Mr. Robinson's Fig. 8 represent merely the relation of silt to rainfall. They do not specify where the silt will originate. Obviously, vegetation is most effective in decreasing erosion of hill slopes, on alluvial fans, and at the head of gullies. After the water is concentrated in the valley floors, a good growth of grass tends to spread this water and keep it spread, by which means silt is brought down more slowly. A decrease in vegetation, particularly grass, whether brought about by overgrazing or a change in climate to greater aridity, induces the formation of channels. Thus, although the general erosion decreases, a special and concentrated stream erosion may ensue.

This type of concentrated stream erosion has within recent years produced channel trenches or arroyos on most ephemeral streams of the Southwest. The cutting of arroyos, or as some call it—the Recent Epi-Cycle of erosion—seems to be due, following the principle brought out in the measurements of silt at Zuñi, to a decrease in the vegetative cover. The delicate balance of erosional forces has been upset and, whereas the streams once aggraded their valleys or were at least neutral, they now are actively degrading them.

That overgrazing was the "trigger pull" which precipitated this accelerated erosion is apparently true, but the underlying cause is doubtless a change to a dryer climate. This change probably occurred before the settlement of the country by white men and there appears to be no good reason to believe that a reversal of the climatic swing is imminent. Sound engineering practice dictates careful silt studies in advance of the building of reservoirs and active measures for the control of silt on the water-sheds of those already built. The successful control of silt depends on (1) reasonable restriction of grazing; (2) introduction of drought-resistant plants that are non-palatable to stock (salt cedar (*Tamarix* sp.,) etc.); and (3) construction of retarding and retaining works.

The success of the works put in on the Zuñi may lead to a tendency to put faith in such direct methods of control to the exclusion of the indirect methods which can only be put into effect after a campaign of public education and with the aid of State authority.

J. C. STEVENS,* M. AM. SOC. C. E. (by letter).†—The author has brought forcibly to the attention of the profession and to the world the seriousness of the silt problem in one of its most menacing forms. The loss of storage capacity, amounting in effect to nullifying large investments in storage works, is a problem the seriousness of which is realized in only a few quarters.

* Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

† Received by the Secretary, June 12, 1928.

What is to become of the civilizations that will grow up as a result of the creation of storage works on the silt-laden streams of the West? Are lands now teeming with life and verdure doomed to revert to sagebrush and cactus? Is science powerless to halt the march of this relentless foe?

It is staggering to think that in so brief a period as 7 years the capacity of a 48 000 acre-ft. storage reservoir was reduced to one-half and that in a subsequent period of 13 years its usefulness has been practically obliterated.

The Special Committee on Irrigation Hydraulics of the Society has chosen the "silt problem" as one of its subjects for study. It is to be hoped that this paper will bring forth timely discussion and rivet the attention of the Engineering Profession on the gravity of this problem.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

UPWARD PRESSURES UNDER DAMS: EXPERIMENTS BY THE UNITED STATES BUREAU OF RECLAMATION

Discussion*

By Messrs. Charles Terzaghi, J. C. Stevens, and F. W. Hanna.

Charles Terzaghi,† M. Am. Soc. C. E. (by letter).‡—Data like those presented in Mr. Hinds' paper are among the most precious that the engineer could possibly obtain because they create a unique opportunity to check the current conceptions against observed facts.

The first group of data concerns dams on a more or less permeable sand and gravel foundation. According to Mr. W. G. Bligh's empirical rule§ a row of sheet-piles should increase the effective distance of percolation by an amount equal to twice the depth of the sheet-piles. During the past years, Mr. Bligh's rule has been also confirmed by both theory and laboratory experiment. Engineers should realize, however, that the theoretical and the empirical rule are only valid when the underground is perfectly homogeneous; that is, when the permeability of the underground is the same in a vertical and in a horizontal sense, and, in Nature, such simple conditions almost never exist. If the underground includes a single layer of feebly permeable material which is penetrated by the sheet-piles, the piling becomes far more effective than it would be otherwise.

Another important question concerns the processes which lead to piping. According to the results of the experimental and theoretical investigations performed by the writer a few years ago concerning the mechanics of piping in uniform sands, the material located beneath the structure remains strictly in

* Discussion of the paper by Julian Hinds, M. Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Associate Prof., Foundation Eng., Mass. Inst. Tech., Cambridge, Mass.

‡ Received by the Secretary, May 7, 1928.

§ "Dams, Barrages, and Weirs on Porous Foundation," by W. G. Bligh, *Engineering News*, 1910, II, p. 708.

equilibrium unless, at some point down stream from the weir, the hydraulic gradient exceeds the critical value, unity.* There is not even a shadow of a possibility for underground erosion. The structure either is safe or else it fails, but there is no possibility for the continued movement of particles. In contrast to this, wherever the underground consists of strata of widely different grain composition, every surface of contact between two different layers represents a zone of weakness. The seepage produced by storage starts an extensive migration of material within the underground. As a result of this migration, the quality of the underground gradually becomes either better or worse, but it certainly does not remain stationary and it may take many years until a state of relative equilibrium is reached. Tables 1† and 2‡ seem to furnish very instructive illustrations for these gradual changes. The results of experimental investigation of the migration of material in stratified underground were recently published by Dr. A. Läufer, in Vienna.§ The factor of safety of a dam against piping depends in a large extent on minor geological circumstances which cannot possibly be anticipated, such as the area and form of lens-shaped beds of more permeable material. Therefore the gradual transformation of the underground on every dam should be watched for a couple of years by means of stand-pipe observations, such as those reported by Mr. Hinds.‖ On some weirs in the Alps that were equipped with a foundation patented by the writer, provisions were made for checking or releasing the flow of seepage according to the results of stand-pipe observations after the weir was built.

Very uncertain, also, are the current assumptions concerning the hydrostatic uplift below masonry dams founded on solid rock. It should be realized that a solid rock foundation with fairly wide groutable fissures may be far less unfavorable than another one, equally solid, with many but very narrow fissures which cannot be grouted. Fig. 24 shows a cross-section through the Oestertal Dam, in Germany, and the distribution of the hydrostatic upward pressure as disclosed by measurements on May 6, 1910, at full reservoir.¶ The upward pressure was measured by means of pressure gauges connected through pipe lines with chambers 4 in. in diameter, arranged along the surface of contact between the dam and the rock. The pressures, *AB* (Fig. 24), were measured along a section where, during construction, the rock appeared to be tight and dry; while the pressures, *AE*, were measured along a section where the rock was somewhat scattered and contained conspicuous water-bearing seams. According to Fig. 24, along the first section, the upward pressure was almost twice as high as it was along the second one. The diagram shows, in addition, how extremely variable the distribution of pressures can be over the base of the same dam.

* Charles Terzaghi, "Der Grundbruch an Stauwerke und seine Verhütung," *Die Wasserkraft*, 1922, p. 445.

† *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 689.

‡ *Loc. cit.*, p. 695.

§ "Lassenbildung und deren Verheilung," *Die Wasserwirtschaft*, 1927.

‖ *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 698.

¶ "Unterdruck bei Staumauern," by R. Schäfer, *Zeitschrift für Bauwesen*, 1913, p. 101.

Still more uncertain are the accepted assumptions concerning the percentage of the area of the base over which the pressure acts. The narrowest seams which can be grouted have a width of about $\frac{1}{20}$ in., while the narrowest seams through which hydrostatic pressure can still be transmitted, have a width of about $\frac{1}{500\,000}$ in. Under the geological conditions found at the site of the American Falls Dam, in Idaho, it seems to the writer that the

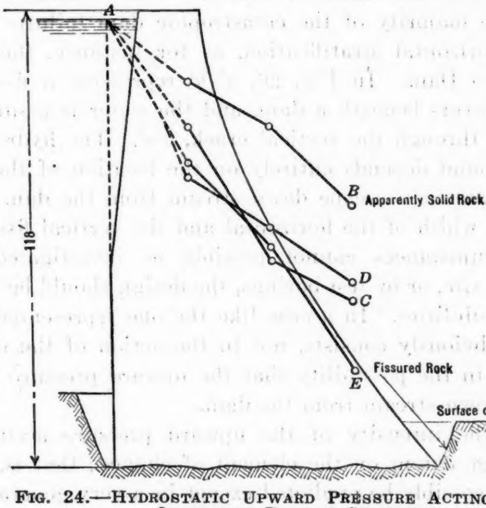

FIG. 24.—HYDROSTATIC UPWARD PRESSURE ACTING ON THE OESTERTAL DAM IN GERMANY.

assumption of the pressure acting over one-third the area might have been fairly conservative. On the other hand, there are geological conditions which present the possibility that the pressure may act over more than four-fifths of the base area. This is particularly true in cases where the valley was carved

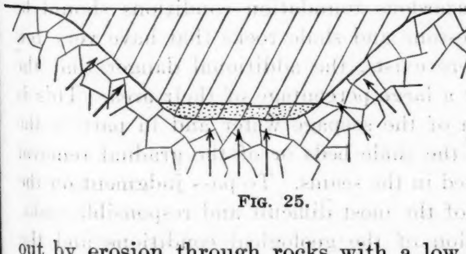

FIG. 25.

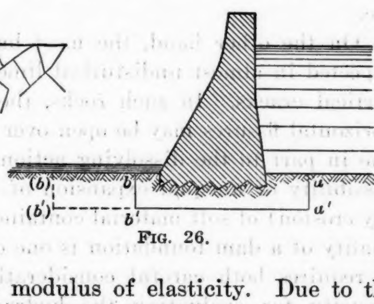

FIG. 26.

out by erosion through rocks with a low modulus of elasticity. Due to the removal (by erosion) of the rock which formerly occupied the shaded area in Fig. 25, the remaining rock underwent unequal elastic expansion associated with the formation of cracks and slight mutual displacement of the rock fragments. As a result of such a displacement, the area of actual contact between the rock fragments may be reduced to a small fraction of the area of the base of the dam, leaving between the rock faces both horizontal and inclined fissures too narrow to yield a visible amount of seepage, yet wide enough to transmit hydrostatic pressure.

Fig. 24 illustrates another great danger. Referring to the pressure line, *AB,* for the Oestertal Dam, the possibility of active hydrostatic uplift occurring within the rock at the down-stream end of the section near *B,* is apparent. This is a danger that is completely absent at the corresponding points (near *E,* Fig. 24) at the Willwood and American Falls Dams. Such possibilities require careful consideration, particularly in cases where the underground consists of horizontal limestone strata or shales. It is certainly more than a mere chance that the majority of the catastrophic dam failures occurred on rocks with almost horizontal stratification, as for instance, the two Austin Dams and the Jumbo Dam. In Fig. 26, a' b' represents a fissure between two horizontal rock layers beneath a dam, and the water is assumed to enter the horizontal fissure through the vertical crack, $a\,a'$. The hydrostatic uplift acting within the ground depends entirely on the location of the crack, $b\,b'$, through which the water can escape down stream from the dam and also on the ratio between the width of the horizontal and the vertical fissures. Since these important circumstances cannot possibly be investigated, either by inspection of the dam site, or by test borings, the design should be based on the most unfavorable possibilities. In a case like the one represented in Fig. 26, the greatest danger obviously consists, not in the action of the uplift on the base of the dam, but in the possibility that the upward pressure will lift the weaker roof of rock down stream from the dam.

Obviously, then, the intensity of the upward pressure acting on dams depends to a very large extent on the element of chance; that is, on circumstances which cannot possibly be predicted except in a very general way. The most dependable rocks are obviously those that contain a moderate amount of irregular groutable fissures, such as those that form the underground of the Willwood and the American Falls Dams. The grout curtain creates a considerable drop of the hydrostatic head at the place where it intersects the current of seepage water; while, down stream from the curtain, the open fissures facilitate the free escape of the seepage water toward the down-stream toe.

On the other hand, the most hazardous foundation conditions should be expected in almost undisturbed limestone and shale rocks that have very few vertical cracks. In such rocks, there exists, the additional danger that the horizontal fissures may be open over a large percentage of their area. This is due in part to the dissolving action of the seepage water and in part to the possibility of unequal expansion of the shale beds or of the gradual removal (by erosion) of soft material contained in the seams. To pass judgment on the quality of a dam foundation is one of the most difficult and responsible tasks. It requires both careful consideration of the geological conditions and the capacity for evaluating the hydraulic importance of the geological facts which can only be obtained by a thorough training in the hydraulics of seepage.

Many textbooks on engineering contain simple and alleged empirical rules for computing the intensity of the hydrostatic uplift acting on the base of dams, irrespective of the geological conditions existing at the dam site. Such rules must always be considered to be dangerous. The young engineer

is led by them to ignore the complexity of the problem and the layman may conclude from the apparent simplicity of the computation that the design of dam foundations does not require any special expertness. A warning should be issued against considering the data concerning the Willwood and the Austin Dams as one more argument for assuming that the present practice for the computation of the hydrostatic uplift is always sound. The warnings which were issued on previous occasions by John R. Freeman, Past-President, Am. Soc. C. E., concerning the same subject were certainly not exaggerated, and every year brings new examples that justify his opinion. Prompted by the results of his pressure observations at the Neyetal Dam, in Germany, R. Schäfer* advised engineers to assume upward pressure distribution according to a straight line, sloping from full head up stream to half head down stream. He recommends assuming that the upward pressure acts over the entire area of the base. Thus, every set of observations may lead to other conclusions. Nevertheless, if placed in the hands of cautious and experienced engineers, data such as those presented by Mr. Hinds are of inestimable value. In Europe, studying the uplift and the seepage conditions of dams continuously until they become stationary has become a fairly common practice; and it would be of great value if the same practice would become a routine custom in the United States.

J. C. STEVENS,† M. AM. SOC. C. E. (by letter).‡—The data presented in this paper are invaluable and will aid greatly in the design of dams as regards uplift pressures. The simplified presentation of the maximum uplift pressures observed, compared with those used in design as in Figs. 15§ and 20‖, is particularly to be commended. These show the pressure areas over the entire base of the dam. The writer believes this is the proper manner to treat such uplift pressures in design and also to present experimental data relative thereto.

The author fails to describe the drain-holes in the American Falls Dam. Presumably, they are independent holes in line with the pressure pipes, *A*, but their spacing and depth are not indicated. Their effect, however, is quite apparent in Fig. 20. It is hardly possible that the pressure lines are straight from the *A*-line to the free-water surface, since this ignores the effect of the cut-off wall. Similarly, it is not likely that the pressure lines are straight between rows of holes or between the *D*-line and tail-water. This shows that while unit uplift pressures may be measured at many points in the dam, engineers are yet almost entirely ignorant of the area to which these unit pressures apply.

This question was considered at length at a recent meeting of the Special Committee on Irrigation Hydraulics of the Society, and steps were taken to inaugurate some research work on this phase of the subject.

It would seem that uplift in the mass of the concrete itself is possible, particularly along horizontal planes between pours and especially when the dam

* "Unterdruck bei Staumauern," *Zeitschrift für Bauwesen,* 1913, pp. 101-118.

† Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

‡ Received by the Secretary, June 12, 1928.

§ *Proceedings,* Am. Soc. C. E., March, 1928, Papers and Discussions, p. 702.

‖ *Loc. cit.,* p. 708.

is loaded so that there is practically no pressure in the masonry itself at the face of the dam.

If dams could be designed so that the pressure in the masonry at all points equals or exceeds the water pressure at that point, uplift would be taken care of automatically. This is impracticable in high dams, however, and uplift pressures must be reduced, first, by adequate cut-off walls and, second, by effective drainage down stream therefrom.

F. W. HANNA,* M. AM. SOC. C. E. (by letter).†—There is probably no other question arising in the design of a dam, on which there is so much guesswork, as on the proper uplift to assume in order to meet actual conditions that occur in the foundation of the structure. This paper, therefore, is timely and valuable in the present period of extensive dam building. The fact that the U. S. Reclamation Service has provided facilities for observing pressures in three different types of foundation materials, is commendable, and additional similar facilities in other foundations are desirable.

The author's contention‡ that filter bed action exists in the stream beds above overflow dams is undoubtedly correct, but it is of relatively small importance in the designs of dams made to withstand the worst uplift conditions. It is of extreme importance, however, when it comes to a proper interpretation of seepage data. The paper brings out this fact clearly. The erodability of the filter film, where the dam is relatively low, is well illustrated by the Colorado River and Percha Dams.

The author shows§ that the data at the Colorado River Dam neither prove nor disprove Bligh's theory of uniform pressure drop along the line of creep, and he properly ascribes this to the irregular foundation conditions at the dam. In this connection it is important to observe that conditions of such uniformity as to give results in conformity with Bligh's theory rarely occur in the foundations of dams. Of the four dams discussed by the author, only one has a fairly uniform foundation throughout. The Colorado River Dam is founded on loose, coarse gravel of varying thickness, with parts of the cut-off wall extending to tighter material. The foundation of the Willwood Dam is partly shale and partly sandstone; and the American Falls Dam rests on columnar basalt. In the foundations of all these dams, there is opportunity for lateral movement of the seepage water due to variations in the depth or character of the materials under them, thus destroying the conformity of pressure creep theory with the Bligh theory. On the other hand, the Percha Dam is founded on rather uniform material as regards horizontal layers parallel to the axis of the dam; and at this project there is a more reasonable conformity with Bligh's theory, as might be expected. The sandstone section of the Willwood foundation also shows reasonable conformity with Bligh's theory.

Another interesting point brought out by the data presented by the author is the distribution of pressure on the base of the dam.‖ In the design of dams

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† Received by the Secretary, June 26, 1928.

‡ *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 685.

§ *Loc. cit.*, p. 688.

‖ *Loc. cit.*, pp. 696 et seq.

it is customary to assume the uplift pressure to act either on all, or on a fractional part, of the base. The lack of uniform distribution of pressure is well indicated by the points, B, in Lines 1 and 2 of the Willwood Dam and the points, C, at Stations 11+10 and 13+00 in the American Falls Dam. The foundation materials of these dams are such as to make these results subject to anticipation. On the other hand the foundation materials of the Percha Dam and the sandstone parts of the foundation of the Willwood Dam are such as to lead to the assumption of uniform distribution. In this connection the experiment* of H. de B. Parsons, M. Am. Soc. C. E., is of interest. In carefully conducted experiments, Mr. Parsons found, for confined clay, sand, and gravel, that the effective area of the base is approximately 100 per cent. This indicates that the pressure is applied to the base, not only by direct water contact, but by the points of contact of the soil particles with the base. This result is for uniform hydrostatic head on the entire base and is not to be confused with the variation of head from heel to toe of dam.

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 941.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

CONTINUOUS BEAMS OVER THREE SPANS

Discussion*

By Messrs. Clyde T. Morris, A. Lincoln Hyde, Harold E. Wessman, and Robert M. Wilder.

Clyde T. Morris,† M. Am. Soc. C. E. (by letter).‡—It is with gratification that one sees the tendency of designing engineers in recent years to calculate stresses with greater precision. This tendency has been especially marked in reinforced concrete design, probably due to the fact that its basic theory is comparatively new and has been developed in connection with tests in university laboratories, by research workers skilled in this type of investigation. The uncertainties of the physical properties of concrete, namely, modulus of elasticity, time yield, and ultimate strength, would not seem to call for as careful theoretical computation of stresses as might be advantageous in the older material—steel—which has well known and constant physical characteristics. In spite of this, steel structures are still designed on assumptions of simple beams, girders, and trusses, connected by hinged joints. There is no doubt in the writer's mind that the steel frame of a modern, well-designed office building is just as much a monolith, as the building with a reinforced concrete frame. In fact, the effect of time yield of concrete upon the negative dead load moments at the joints of a reinforced concrete frame must be to relieve them to a considerable extent and to increase the positive moments at the mid-spans.

Therefore, it would seem that careful consideration of continuity in a steel frame with well-designed joints is just as important as in a reinforced concrete frame. Usually, the joints of steel frames must be designed to resist wind moments, and if properly designed will also care for the joint moments caused by dead and live loads.

* Discussion on the paper by I. Oesterblom, M. Am. Soc. C. E., continued from May, 1928, *Proceedings*.

† Prof. of Structural Eng., Ohio State Univ., Columbus, Ohio.

‡ Received by the Secretary, March 19, 1928.

It has been the writer's practice, in designing steel frame buildings, to make use of the same diagrams and methods developed for calculating the moments at the joints of rigid frames, which are published in the various texts on reinforced concrete construction.

Mr. Oesterblom illustrates well the fallacy of designing by rule-of-thumb methods. The writer would go even further and take into account the restraining effect of the supports. It is seldom that a continuous beam is supported on joints which are free to turn. It is usually rigidly connected to the columns, and the bending moments in both must be calculated and the design proportioned accordingly.

In a bent, with equal or nearly equal story heights, the slope deflection equations for any one floor may be written independently of the other floors, by assuming that the angles of rotation of the joints immediately above and below are equal to those at the floor under consideration. This is equivalent to assuming a hinged-end condition in all the columns at the mid-story heights. In a symmetrical three-span bent, such as the author treats, this results in two equations and two unknown slopes when the loads are symmetrical, as they are for maximum span moments and for dead load.

A. LINCOLN HYDE,* M. AM. SOC. C. E. (by letter).†—The author is to be commended for his efforts to simplify, for practicing structural engineers, the frequently occurring problem of continuous beams over three spans, symmetrical with respect to end spans, but with the middle span varying from 0.2 to 2.5 times the end span. The limits of his values of x‡ seem to be well chosen. For office buildings, hotels, hospitals, etc., the values of x will fall well below 1.0, but for highway bridges in locations where it is desired to provide wide middle channels, the values of x may approach 2.5.

The writer feels that some designers will be saved considerable work by the use of the author's table of moment factors (Table 3‡), but designers of reinforced concrete are vitally interested in maximum shears and in the location of inflection points as well as in maximum bending moments.

For his own use the writer has prepared tables of reaction coefficients for beams continuous over three spans, the end spans being equal, and the middle span varying from 0.2 to 2.5 times the end span. With these tables the designer may, with little labor, compute maximum bending moments and maximum shears, and may also locate inflection points. Tables 5 and 6 are herewith submitted with the hope that they may supplement Table 3, and prove of value to other designers. They were developed from the well-known Merriman formulas for a symmetrical span arrangement; x is the ratio of the middle spans, and all coefficients are computed on the basis of outside spans.

To find any reaction, first find the ratio, x, of the middle span to the end span, and then multiply the reaction coefficient, θ, by $w\,l$. The value, w, must be the dead load or the live load per linear foot as the case may be, and the value, l, must be the length of the end span, in feet. The dead and live load effects must be added for total effect.

* Prof. of Bridge Eng., Univ. of Missouri, Columbia, Mo.

† Received by the Secretary, March 22, 1928.

‡ *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 719.

TABLE 5.—Reaction Coefficients in the Formula, $R = \theta\, w\, l$.

Values of Coefficient, θ.

Full Uniform Load.			Uniform Load, Middle Span.		
Values of x.	R_1 and R_4	R_2 and R_3.	R_1 and R_4.	R_2 and R_3.	Values of x.
0.2	0.4031	0.6969	−0.0007	0.1007	0.2
0.3	0.4115	0.7385	−0.0022	0.1522	0.3
0.4	0.4169	0.7831	−0.0050	0.2050	0.4
0.5	0.4196	0.8304	−0.0090	0.2590	0.5
0.6	0.4200	0.8800	−0.0142	0.3142	0.6
0.7	0.4181	0.9319	−0.0209	0.3709	0.7
0.8	0.4140	0.9860	−0.0291	0.4291	0.8
0.9	0.4080	1.0420	−0.0388	0.4888	0.9
1.0	0.4000	1.1000	−0.0500	0.5500	1.0
1.1	0.3900	1.1600	−0.0628	0.6128	1.1
1.2	0.3782	1.2218	−0.0771	0.6771	1.2
1.3	0.3645	1.2855	−0.0931	0.7431	1.3
1.4	0.3490	1.3510	−0.1106	0.8106	1.4
1.5	0.3317	1.4183	−0.1298	0.8798	1.5
1.6	0.3126	1.4874	−0.1506	0.9506	1.6
1.7	0.2918	1.5582	−0.1730	1.0230	1.7
1.8	0.2692	1.6308	−0.1970	1.0970	1.8
1.9	0.2448	1.7052	−0.2227	1.1727	1.9
2.0	0.2158	1.7812	−0.2500	1.2500	2.0
2.1	0.1909	1.8591	−0.2789	1.3289	2.1
2.2	0.1614	1.9386	−0.3095	1.4095	2.2
2.3	0.1301	2.0199	−0.3418	1.4918	2.3
2.4	0.0972	2.1028	−0.3756	1.5756	2.4
2.5	0.0625	2.1875	−0.4111	1.6611	2.5

TABLE 6.—Reaction Factors in the Formula, $R = \theta\, w\, l$.

Values of Coefficient, θ.

Uniform Load, Left-End Span.					Uniform Load, Both End Spans.		
Values of x.	R_1.	R_2.	R_3.	R_4.	R_1 and R_4.	R_2 and R_3.	Values of x.
0.2	0.3951	1.1731	−0.5769	0.0087	0.4038	0.5962	0.2
0.3	0.4025	0.9598	−0.3735	0.0112	0.4137	0.5863	0.3
0.4	0.4089	0.8516	−0.2735	0.0130	0.4219	0.5781	0.4
0.5	0.4143	0.7857	−0.2143	0.0143	0.4286	0.5714	0.5
0.6	0.4190	0.7412	−0.1754	0.0152	0.4342	0.5658	0.6
0.7	0.4232	0.7091	−0.1481	0.0158	0.4390	0.5610	0.7
0.8	0.4269	0.6847	−0.1278	0.0162	0.4431	0.5569	0.8
0.9	0.4303	0.6655	−0.1123	0.0165	0.4468	0.5532	0.9
1.0	0.4333	0.6500	−0.1000	0.0167	0.4500	0.5500	1.0
1.1	0.4361	0.6372	−0.0900	0.0167	0.4528	0.5472	1.1
1.2	0.4386	0.6265	−0.0818	0.0167	0.4553	0.5447	1.2
1.3	0.4409	0.6174	−0.0750	0.0167	0.4576	0.5424	1.3
1.4	0.4430	0.6095	−0.0691	0.0166	0.4596	0.5404	1.4
1.5	0.4450	0.6026	−0.0641	0.0165	0.4615	0.5385	1.5
1.6	0.4469	0.5965	−0.0597	0.0163	0.4632	0.5368	1.6
1.7	0.4486	0.5911	−0.0559	0.0162	0.4648	0.5352	1.7
1.8	0.4502	0.5863	−0.0525	0.0160	0.4662	0.5338	1.8
1.9	0.4517	0.5820	−0.0495	0.0158	0.4675	0.5325	1.9
2.0	0.4531	0.5781	−0.0469	0.0157	0.4688	0.5312	2.0
2.1	0.4544	0.5747	−0.0445	0.0154	0.4698	0.5302	2.1
2.2	0.4557	0.5714	−0.0423	0.0152	0.4709	0.5291	2.2
2.3	0.4569	0.5684	−0.0403	0.0150	0.4719	0.5281	2.3
2.4	0.4580	0.5657	−0.0385	0.0148	0.4728	0.5272	2.4
2.5	0.4590	0.5632	−0.0368	0.0146	0.4736	0.5264	2.5

Application of Reaction Coefficients.—Using the data of the author's Example 1,* and dealing with Span 12: For full uniform dead load, and uniform live load over both end spans,

$$R_1 = 0.417 \times 67 \times 22.5 + 0.4218 \times 60 \times 22.5 = 630 + 570 = 1\,200 \text{ lb.}$$

The section of zero shear and maximum bending moment is distant to the right of Support 1,

$$\frac{630 + 570}{67 + 60} = 9.45 \text{ ft.}$$

The maximum bending moment is,

$$M_{12} = 1\,200 \times 9.45 - 127 \frac{(9.45)^2}{2} = 5\,670 \text{ ft-lb.}$$

which checks the author within 1 per cent.

The inflection point is distant to the left of Support 2,

$$22.5 - 2 \times 9.45 = 3.60 \text{ ft.}$$

The shear at Support 1 is 1 200 lb., and at the left of Support 2, it is,

$$1\,200 - 22.5 \times 127 = -1\,660 \text{ lb.}$$

For Span 23 the loading for maximum bending moment is the same as before. The reaction at Support 2 is,

$$R_2 = 0.783 \times 67 \times 22.5 + 0.5871 \times 60 \times 22.5 = 1\,960 \text{ lb.}$$

and,

$$M_{23} = 1\,200 \times 27 + 1\,960 \times 4.5 - 127 \times 22.5 \times 15.75 - 67 \times 4.5 \times 2.25 = -4\,470 \text{ ft-lb.}$$

which checks the author closely.

The bending moment being negative over the entire middle span, there will be no inflection point between Supports 2 and 3. The shear at the right of Support 2 is,

$$1\,200 + 1\,960 - 2\,860 = 300 \text{ lb.}$$

The maximum bending moment at Support 2 occurs under the full uniform dead load, and uniform live loads over Spans 12 and 23. The reaction at Support 1 for this loading is,

$$R_1 = 630 + 0.4089 \times 60 \times 22.5 - 0.0050 \times 80 \times 22.5 = 1\,170 \text{ lb.}$$

and,

$$M_2 = 1\,170 \times 22.5 - 127 \times 22.5 \times 11.25 = -5\,820 \text{ ft-lb.}$$

which again checks the author closely.

Harold E. Wessman,† Jun. Am. Soc. C. E. (by letter).‡—This and other recent papers§ relating largely to moments in continuous spans are focusing the attention of structural engineers more and more on "statically indeterminate" structures. Scientific research has been stimulated and means are

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 719.

† Instr., Structural Eng., Dept. of Civ. Eng., Univ. of Illinois, Urbana, Ill.

‡ Received by the Secretary, April 23, 1928.

§ "Moments in Restrained and Continuous Beams by the Method of Conjugate Points", by L. H. Nishkian and D. B. Steinman, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 1; and "Distribution of Reinforcing Steel in Concrete Beams and Slabs," by Boyd S. Myers, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 174.

being sought to simplify methods of analysis, so that the designer in continuous structures may determine his unknown forces without going through long, involved, tedious, and costly computations. There has been a tendency in some circles to allow for the unavoidable presence of continuity by large factors of safety or arbitrarily selected coefficients, which have been established by well-meaning authorities for use in specific cases only. The writer believes that within the next few years there will be radical developments in this field of analysis, which will not only check the tendency to shroud the so-called "indeterminate" structure in a cloak of mystery, but will present it in such a light that design practice will know it better and use it more intelligently. Some engineers apparently believe that "statically indeterminate" structures are "absolutely indeterminate" structures and cannot be analyzed by any means.

Mr. Oesterblom, however, makes a rather sweeping indictment of universities, municipal authorities, and consulting engineers on what he states is their current practice in the teaching of design or the actual design of continuous structures. No doubt there are cases warranting his indictment, but general conclusions are not justified. The writer has been connected with all three groups mentioned by the author and knows that representatives of each* are facing the facts of design for continuous structures as they exist and are not adhering blindly to those conventional rules found particularly in reinforced concrete design codes and which apply only in a limited number of cases.

The author takes the familiar three-moment equation in a form similar to that presented by Clapeyron and from it derives expressions for the moments at the critical points in a three-span system due to loading one span at a time with a unit uniform load per foot. The two outside spans are alike and of length unity and the center span is of variable length, x. The critical points are at the supports and at or near the center of each span. For the system outlined, the three-moment equation is not particularly burdensome and lends itself readily to the computation of a table such as the author makes (Table 3†); but when the systems are decidedly unsymmetrical, when they have three or more spans, and involve variable moments of inertia, this method becomes very complex and tedious. Moreover, when the beams or girders are framed into restraining columns or walls at the supports, it cannot ordinarily be used at all. The writer is familiar with several methods for computing moments and shears in continuous systems. The one which he prefers, is extremely flexible, may be readily applied regardless of the number of spans, varying moment of inertia, or restraint at supports, and gives accurate results with remarkable rapidity. This method has been used in checking the author's Table 3 and in obtaining results which follow.

The author loads one span at a time and computes "moment factors" for the point of maximum moment in the loaded span, for the mid-points of the unloaded spans, and for the supports. In the middle span, the point of

* "Structural Design of Bridges in Grant Park," by C. R. Hoyt, Assoc. M. Am. Soc. C. E., *Journal*, Western Soc. of Engrs., October, 1925; and, "The South Park Boulevard Viaduct," by T. L. Condron, M. Am. Soc. C. E., *loc. cit.*, July, 1923.

† *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 719.

maximum positive moment is the center of the span; but in each outside span when loaded, it is approximately at 0.41 from the support instead of at the center. For the effect in the same span due to loads on the other spans, the author computes the moment ordinate, not at the same point where his maximum positive moment occurred, but at the center of the span. For the combined moment due to loading all the spans, he consequently adds moment ordinates which do not occur at the same point. An inspection of Fig. 2* will make this clearer. This procedure is somewhat inconsistent, but the author justifies it by stating that, "on all the spans the lines representing the two unloaded effects are compensating, and the resultant of the two is almost horizontal." That is only true for the outside spans when $x < 0.60$. (See Table 3.) When $x > 0.6$, the secondary $-M$ in the first span, due to the second span load, becomes very much greater than the secondary $+M$ in the first span due to the third span load.

The total combined moment is not exact (as the author states), but it is close enough for practical purposes, being 1% in error when the ratio of center span to end span is 0.5 and about 4% in error when all the spans are equal in length. If all ordinates had been determined at the mid-points, the error would vary within nearly the same range.

Moreover, the author's coefficients are based on the assumption of knife-edge supports. Even in slabs and beams not framed into columns or deep girders, the actual conditions give area support rather than line support. In addition, they develop some restraint, the effect of which is to reduce $+M$ in the center of the span and in some cases the $-M$ at the support.

Those responsible for the codes of the Joint Committee on Specifications for Concrete and Reinforced Concrete, both old and new, and the American Concrete Institute Code may have had this effect in mind when they specified the clear span as the effective length to be used in determining moments. Definite information on this point apparently is hard to obtain.

In view of the foregoing, the writer believes that the author could have saved himself some work by computing the span moments at the mid-points only and at the supports. The results are accurate enough for those practical purposes for which the author's moment factors may be used, namely, the design of three-span slabs, and beams symmetrical, or approximately so, about the center span and with little or no restraint at the supports.

The author illustrates† the use of his table of moment factors (Table 3) by application to a typical design problem, a three-span system with a 9-ft. corridor span and two outside spans, 22 ft. 6 in. each. After finding total maximum positive and negative moments for dead load over all spans and moving uniform live load, he compares them with the moments obtained by using the conventional coefficients, $\frac{1}{10}$ and $\frac{1}{12}$, which were never intended to apply to anything but equal spans. Naturally, there is a large discrepancy in the two results. (See Table 4‡.) The author points to this as a very

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 715.
† *Loc. cit.*, p. 719.
‡ *Loc. cit.*, p. 723.

common case in which moments are most erroneously calculated. The writer feels that the case is the exception rather than the rule, for he doubts very much that the average designer would try to apply the conventional coefficients for equal spans to systems similar to that of the author, in which the center span is so much shorter than the end spans. The picture of the physical action of the short center span would be apparent enough to most designers to warn them of high negative moment at the supports and the possibility of negative moment over the entire center span.

The author calls attention* to the value of the moment factors in showing the effect on any critical point due to loading in any one span and how certain span loads must be omitted to obtain maximum effects. The writer would like to call attention to the value of qualitative influence lines for determining the placing of live loads to cause greatest effect at the points to be considered. For those to whom the concept of the influence line as the shape of the deformed structure when the particular effect is acting at the point under investigation is familiar, the sketching of the shape of the influence line is only a moment's task. The positive and negative areas under the influence line then indicate the position of live loads to give maximum positive and negative effects.

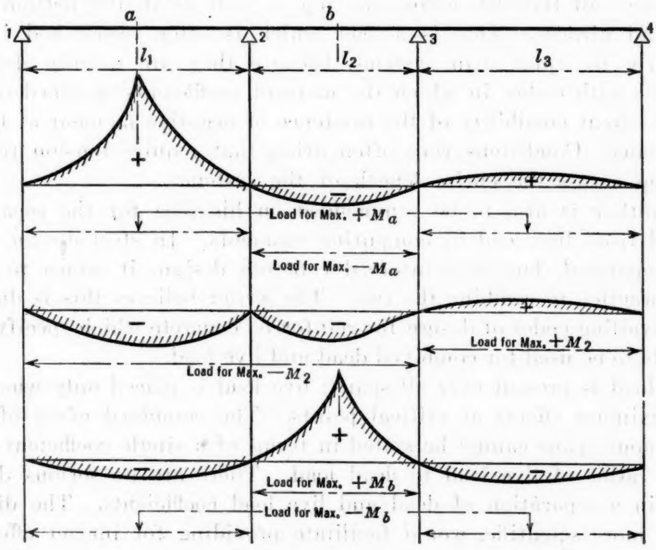

FIG. 3.—INFLUENCE LINES FOR MOMENTS AT CRITICAL POINTS.

Fig. 3 shows influence lines for moments at various governing points in a three-span system. Loading conditions for maximum positive and negative effects are also indicated. For the three-span system, positions of loading are apparent to some designers without drawing influence lines, but where there are more than three spans, and especially where there is multiple-bent or multi-story framing, qualitative sketching of influence lines is of great value.

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 720.

In most cases, only the sketch is needed; but, occasionally, where maximum moments or shears due to a small number of heavy moving concentrated loads is wanted, it may be advisable to determine the actual influence ordinates, in other words, to find the quantitative influence line. It is a matter of incidental interest to note that the author's moment factors are measures to some scale of the area beneath the influence lines, the scale depending on the ratio of span lengths.

The author is to be commended for calling attention to the possible existence of negative moments over the entire center span, but he sets his point of danger too low. He states,*

"Indeed, the center spans are very severely punished by the end span loads, and the span ratio has to reach 0.60 before the danger of negative moments is avoided."

The author overlooks the effect of varying the ratio of $\frac{\text{live load}}{\text{dead load}}$. This does not have to be large or unusual to cause negative moment over the entire center span, even when the center span is greater in length than the outside spans. Moreover, as the span ratio becomes greater than 1.0, negative moment may occur along the entire length of the outside span, necessitating tension steel all the way across the top as well as in the bottom of slabs, beams, and girders. This is a fact which is often overlooked in design, particularly in equal span systems, because they are usually designed in accordance with codes, in which the moment coefficients specified completely ignore the great possibility of the existence of negative moment at the center of the spans. Conditions very often arise, that require tension reinforcing in the top across the entire length of the system.

The author is also to be commended on his plea for the separation of dead load from live load in computing moments. In steel design, they are usually separated, but in reinforced concrete design, it seems to be quite general practice to combine the two. The writer believes this is due largely to the prevailing codes of design for reinforced concrete which specify moment coefficients to be used for combined dead and live load.

Dead load is present over all spans; live load is placed only where it will cause maximum effects at critical points. The combined effect of the two in continuous spans cannot be stated in terms of a single coefficient that will cover all ratios of live load to dead load. There are no serious difficulties involved in a separation of dead and live load coefficients. The distinction would be more scientific, would facilitate providing for impact effects when necessary, as a simple proportion of live load effects, and would promote economy of materials, especially in systems subject to a fixed loading that extends over all spans.

The chief value of the author's paper is in the remarks calling attention to the defects in most of the codes for the design of reinforced concrete. Such defects invite careful consideration of a situation fraught with some elements of danger. There is a pronounced tendency to restrict and guide design practice in reinforced concrete by an apparent completeness of rules

* *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 718.

that will allow no man to go astray in his design. It is a tendency which, on the one hand, removes the premium for technical ability and judgment, making all designers day laborers, and on the other hand, invites danger due to the fact that the rules are not actually complete and that they ignore actual conditions which may exist in cases to which the rules are usually applied. The reinforced concrete design codes would be greatly improved if they were patterned more closely after existing specifications for steel design which do not attempt to "corral" structural analysis.

ROBERT M. WILDER,* JUN. AM. SOC. C. E. (by letter).†—The writer thoroughly agrees with the author as to the danger of neglecting such important factors as variability in span lengths, loads, and sections. The author has very ingenuously evolved a table (Table 3‡) to facilitate the calculation of moments in continuous beams with due regard to these factors.

One factor which influences the moments in beams just as certainly, has been entirely neglected; that is, the restraint offered at the support to angular deflection of the beam at that point. This restraint may be due to the supporting columns in the case of beams, or to the supporting beams in the case of slabs.

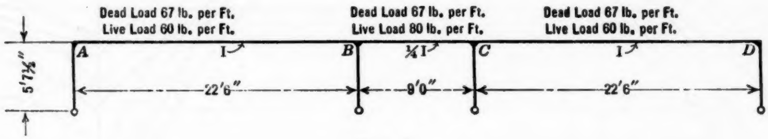

FIG. 4.

This restraint is seldom absent where there is any continuity, especially in buildings, the field which the author rightly stressed as being particularly subject to "handbook" methods of design.

TABLE 7.—MAXIMUM MOMENTS IN FIG. 4, DUE TO VARIOUS DEGREES OF COLUMN RESTRAINT, IN FOOT-POUNDS.

Maximum moment at point.	COLUMN RESTRAINT OR MOMENT OF INERTIA OF COLUMNS DIVIDED BY MOMENT OF INERTIA OF END BEAMS.			
	0*	$\frac{1}{8}$	$\frac{1}{2}$	∞
A (toward *B*)	0	2 080	−3 870	−5 360
Center *A B*	±6 800	±5 200	±3 880	±2 680
B (toward *A*)	−3 540	−4 200	−4 800	−5 360
B (" *C*)	−3 540	−2 680	−1 830	− 990
Center *B C*	−2 010	−1 200	−160, or ±220	± 500

* No column restraint.

To determine the actual effects of column restraint, the writer used the same illustration presented by the author, added various conditions of column restraint, and calculated the maximum moments. This case is shown in Fig. 4. It might be noted that within the limits of practical computation the

* Cuban Representative, Fred T. Ley & Co., Inc., Havana, Cuba.

† Received by the Secretary, May 1, 1928.

‡ *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 719.

results would have been the same with columns above as well as below, their moments of inertia being one-half those used in the case of columns below only. The results of the calculations are given in Table 7.

The results are shown graphically in Fig. 5. The upper curve in each case is for loadings producing maximum moments; the lower, minimum moments. Only one-half of the three spans is shown, as the other half would be symmetrical.

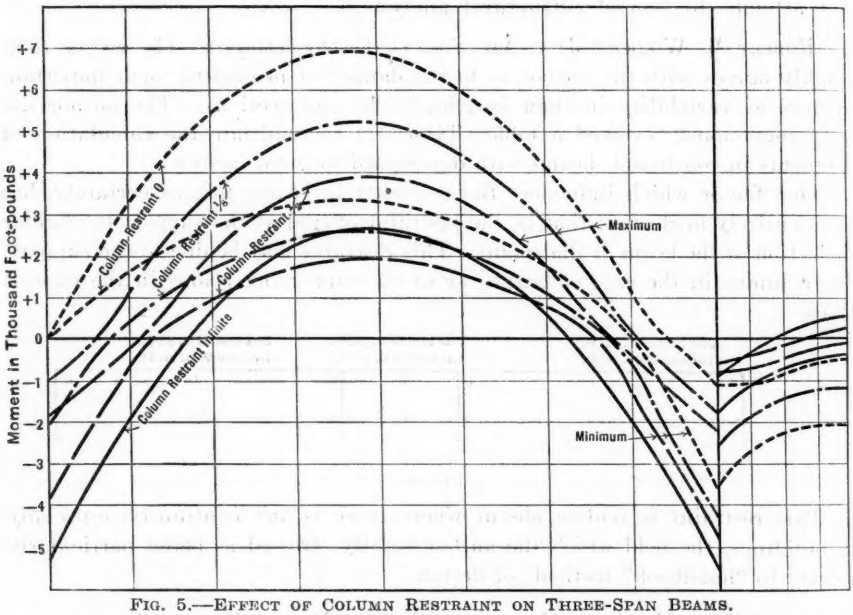

Fig. 5.—Effect of Column Restraint on Three-Span Beams.

The cases of restraint used are quite common in any building and, in most instances, the values would be between the nominal value, $\frac{1}{2}$, and infinity. It can thus be seen that neglecting the effect of column restraint is quite as dangerous as neglecting the other factors.

In the case of concrete construction still another factor presents itself, which was not noted in this paper, namely, the effect of moments of inertia varying from center to support in the same beam.*

The arbitrary moment factors used in so many specifications and building codes are admittedly intended only for certain general conditions of spans, loads, and sections. Where the actual conditions depart from these general conditions to such an extent that the code cannot be modified by ordinary judgment, the designer should make a careful analysis of the structure, taking into consideration not only the factors of continuity, span, load, and section, but also such important factors as restraint at supports and varying moments of inertia in the same span.

The author is to be commended for calling attention to the need of revising current practice in this field.

* The effect of this factor was shown by Mr. R. E. Spaulding in an able article in *Engineering News*, January 13, 1910.

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PAPERS AND DISCUSSIONS

HYDROSTATIC UPLIFT IN PERVIOUS SOILS

Discussion*

By Messrs. F. R. Harris, J. C. Meem, Eugene E. Halmos, T. R. Lawson, J. Vipond Davies, Lazarus White, T. Kennard Thomson, Daniel E. Moran, Alexander V. Gallogly, Allen Hazen, Charles Terzaghi, Frank S. Bailey, Charles R. Gow, and E. L. Chandler.

F. R. Harris,† M. Am. Soc. C. E.—The speaker notes that in his "Introduction",‡ Mr. Parsons states that there is little information on record regarding hydrostatic uplift in pervious soils and was impressed with this as far back as 1909. However, in connection with the design and construction of Dry Dock No. 4 at the Brooklyn Navy Yard, there was considerable discussion on this subject.

It is apparent that a dry dock structure is one in which the hydrostatic uplift is of primary importance. In fact, it is the outstanding condition which influences the entire design of the work. In all dry dock structures built previous to 1909, with which the writer was familiar, the full hydrostatic pressure incident to extreme high tide, or even in excess of this, if the apparent groundwater elevation was higher than extreme high tide, was calculated. There was considerable discussion at that period as to whether or not such uplifts were present and active. One of the theories in vogue then, which Mr. Parsons mentions, is that this pressure was lessened by a reduction in the area of the sub-grade surface in contact with water and subject to water pressure; that is, the surfaces of the subsoil in contact were thought to consist of fine particles of materials pressing against the under side of the structure which served to reduce the area subject to water contact and hydrostatic pressure. It was considered that this, in turn, reduced the total hydrostatic uplift and that this condition obtained so long as the weight of the structure was in excess of such uplift.

* This discussion (of the paper by H. deB. Parsons, M. Am. Soc. C. E., published in April, 1928, *Proceedings*, and presented at the meeting of May 2, 1928), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Rear-Admiral, U. S. N. (*Retired*); Cons. Engr., New York, N. Y.

‡ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 941.

The speaker had some doubts as to the correctness of this theory, in fact, he did not agree with it and, therefore, the dry dock was designed to take full hydrostatic uplift. For future information, devices for recording the actual uplift were placed in the dock floor at several places. These consisted of cylinders with a lead diaphragm, the under side of the diaphragm being at subgrade. The cylinder projected up into the concrete floor and was embedded in it. A plunger or piston with sufficient clearance so as not to bind in the cylinder, was held against the upper side of the diaphragm by a shaft with a capstan screw. On completion of the dock structure observations were made on the uplift, the piston being released for this purpose. In every case practically full hydrostatic pressure was recorded on the diaphragm. In connection with these observations there was provided, through the dock floor, pipes fitted with pressure gauges and the observed water pressure on the gauges was checked against ground-water and tidal observations, the gauges indicating full hydrostatic pressure, fluctuating with tidal changes.

Observations on the diaphragm followed the pressure-gauge observations on the pipe connections so closely that the writer was convinced that the hydrostatic uplift was 100 per cent.

The speaker regrets that it has not, as yet, been possible to arrange these data in a suitable form for publication. The results were, nevertheless, very convincing, in indicating undoubted full hydrostatic uplift.

J. C. MEEM,* M. AM. SOC. C. E.—The author is to be commended highly for the painstaking care with which he has endeavored to solve one of the difficult problems of applied engineering. The speaker has made somewhat similar experiments, although not, perhaps, so exhaustively, with similar apparatus, and has also co-operated with the author, at least in the early stages of this experimental work. He believes, however, that the results of the author's experiments are not conclusive as to the reduction of area of uplift pressure through soil.

The principal reason for this belief is that in order to determine the reduction of area definitely, it is necessary to establish a definite area of contact, and this contact does not appear to have been established in any of the author's experiments, due to the small area of the can or piston, compared with the relative size of the grains of sand or gravel, and to the shallow depth—especially in the case of the hydraulic piston—at which it was subjected to pressure. For instance, under a microscope this piston would have appeared as if resting on a few pinnacles rather than on a flat area. Furthermore, if the author was unable to get more than 15% contact (see Table 4†) between the bottom of the piston and the botton of the container, it is reasonable to suppose that he would not be able to establish a larger area of contact on loose, irregular particles. In the four experiments which the speaker made with hydraulic apparatus similar to that used by the author, he was able to establish a much higher percentage of contact by plunging

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† *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 950.

the piston into the sand, pressing it well down, and leaving it for varying intervals until normal conditions resulted.

In order to give a better understanding of this experiment, a section of the apparatus is shown in Fig. 7(*b*). It consists of a hydraulic chamber with the piston actuated hydraulically by a pump connected by copper pipe to the chamber and carrying a gauge reading to pounds. It was found that it took about 4 lb. of pressure to raise the piston when plunged into water alone.* A table was then placed in the chamber so that sand could be packed around the piston, leaving the bottom free, and it was found that the additional pressure required to raise the piston did not register on the gauge; and frictional resistance due to this sand was assumed to be negligible or less than 25 lb. per sq. ft.

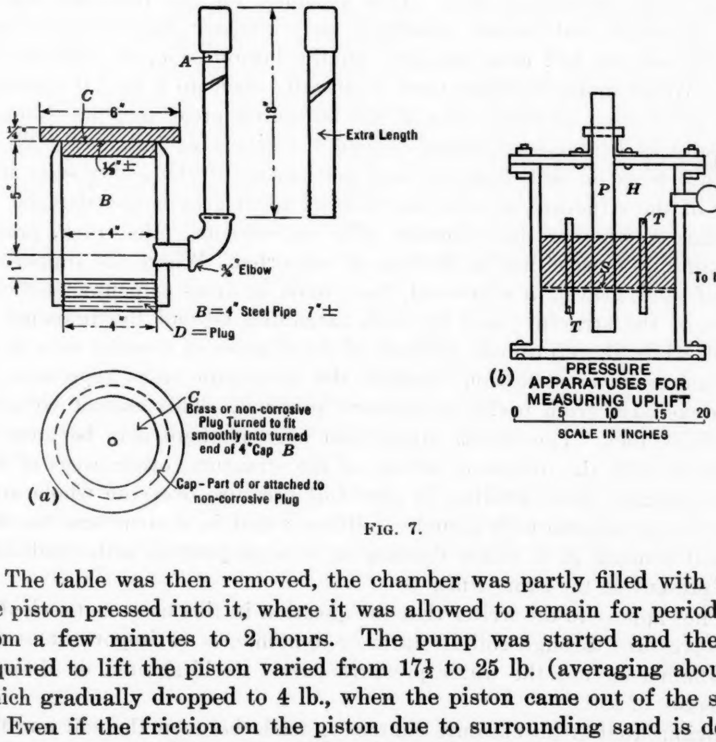

FIG. 7.

The table was then removed, the chamber was partly filled with sand, and the piston pressed into it, where it was allowed to remain for periods varying from a few minutes to 2 hours. The pump was started and the pressure required to lift the piston varied from $17\frac{1}{2}$ to 25 lb. (averaging about 22 lb.), which gradually dropped to 4 lb., when the piston came out of the sand.

Even if the friction on the piston due to surrounding sand is determined by using the coefficient found by the author, it is seen that the pressure required to lift the piston from sand with which it has made contact is three or four times that required to lift it in clear water.

It should be noted that if there is 100% contact of water or pressure on the base of the piston there should also be 100% of water pressure around the piston; in which case, friction should be negligible under the author's theory. When a piston rests in the sand, or is only slightly depressed into it, the reaction of the pressure from the piston to the sand voids under it tends

* *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 367.

to depress the sand further and the action is wedge-like around and under its edges, gradually depressing the sand away from the piston until it has established an area sufficient to lift it. Proper contact may be established in many ways, but those most nearly approximating conditions of practice are: Plunging the piston to a sufficient depth and pressing it firmly into the sand; placing a layer of cement mortar between the piston bottom and the sand; or placing a layer of stiff clay, so that in the latter two cases the action of the water will be through the voids of the sand or gravel against the mortar or clay, raising the latter and, consequently, raising the piston. Careful distinction should be made between structures resting on soil and those resting partly on piles or caissons. In the latter cases the soil may tend to settle away from the structure and leave the area not supported exposed to the full pressure over the remaining area. This accounts for the fact that gauges through submerged and buried structures may register full pressure, even if there may not be full area pressure, on the bottom, top, or sides of the structure. Water under pressure must eventually open up a lead if openings or outlets are caused to exist, even if the adjoining areas may be excluded from pressure by contacts extending beyond the sphere of influence. Where the structure rests on soil, however, any settlement of the soil results in a settlement of the structure, so that the ratio of uplift area is not changed.

Nevertheless, whether the structure rests on caissons, piles, rock, gravel, or soft ground, it must either be floating or supported. Where the supporting areas are of sand, gravel, or other soil, there must be areas of contact between the bottom of the structure and the soil, extending beyond the influence of uplift, and, while the minimum sections of these areas of contact may be in various planes, the cross-section through the minimum areas represents in sum those excluded from uplift or buoyant pressure. This cannot be other than a definite fact. The author states that even if there may be areas of soil in contact with the piston or bottom of the structure, other areas of soil are pressed against these, tending to give full pressure over the whole area. This is not in accordance with actual conditions; that is, a structure together with the soil beneath it, is either floating or it is supported; both conditions cannot be present at the same time.

It does not appear to the writer that the question in this case, as to whether or not the pressure through soil is static or dynamic, is as important as the question whether or not the water pressure exerts itself against or through compact masses of soil.

It is obvious that if an absolute contact is made between the soil and the hydraulic cap (in the experiment referred to), the problem of relative uplift can be solved in the laboratory as well as in the field.

This, however, is impractical with the water standing at or above the gravel or sand level, and the writer conceived the idea of imposing a layer of moist sand between the water-bearing gravel and the cap or piston, with the result already noted.

The speaker has recently made some additional experiments which may be noted here. Referring to Fig. 7(*a*), he has constructed, through the courtesy of Mr. J. Moore, of Richard Dudgeon, Inc., a 4-in. circular chamber

about 5 in. long, with a hydraulic cap and pipe connected by an elbow with the chamber. The cap was ground to a loose hydraulic fit. On pouring water into the pipe it was found that, with the water standing 0.55 ft. above the top of the cylinder, the cap "floated". This was equivalent to lifting a 2.8 lb. weight, the bearing surface of which was 4 in. in diameter. Although the weight of the cap had been computed roughly at more than 3 lb., it was weighed and found to check exactly at 2.8 lb. The chamber was then partly filled with gravel to within ¾ in. of the top, and a layer of sand was put in and covered with a layer of clay or putty, it being desired to show what column of water was required to lift the cap, together with the clay, or both clay and sand. Three trials, which checked closely, showed this column to be between 1.1 and 1.2 ft. Allowing approximately 0.1 ft. for the weight of the sand, a column of 1.2 ft. would be exactly double the clear water column, showing 50% reduction of area due to the gravel.

In some of the later experiments with the apparatus shown in Fig. 7(*a*), there was 4 in. of sand below the cap. It would require a water column of 6 in. approximately to lift this, and about 6 in. more to lift the cap, that is, 1 ft. in all. It is not believed that friction need be considered in so small a layer of soil, with the inside faces greased, and it is thus seen that, if the water stood at 8 in. above the top of the chamber, plus the 4 in. below, this 1-ft. head should lift the sand and cap, or that a 6-in. head above the chamber should lift the cap alone, from clear water.

As a minimum of 1 ft. 8 in. was required to do this, it would seem to indicate a reduction of the pressure area of 70% on the cap alone, or 50% on the cap and sand; that is, if the cap alone had lifted from the sand, it would appear that the pressure acted through 30% of the voids of the sand, or through 50% of the gravel voids in lifting the sand cap—which is almost certainly what happened.

Since these experiments were made, the speaker's assistant, J. C. Fisher, Jun. Am. Soc. C. E., has made several additional experiments. His findings are as follows: When the water-bearing gravel was placed within 1 in. of the top of the chamber and ½ in. of moist sand was placed over it, covered to the top by a clay cap, with the inside of the chamber greased to prevent friction and leakage, this clay cap was lifted (apparently) by a column of water 0.165 ft. above its bottom plane. The column required to lift from clear water was approximately ¾ in., so that 1⅞ in. would show 60% reduction of area pressure, or 40% effective area pressure. This experiment was repeatedly made with approximately similar results. If, as may have been the case, the sand lifted from the gravel, then the column of water, acting ½ in. lower, would have been approximately 0.20 ft., or 2.4 in., of which 1½ in. was required to lift sand and clay. This would show 40% reduction of pressure area, or 50% of pressure area effective. Mr. Fisher thinks, however, that the water acted through the thin layer of sand against the clay cap.

He also found that a can forced into a thin bed of mortar and pressed into sand to insure contact, would not rise (after setting) until its buoyancy was approximately doubled by drawing off the inside water.

The writer believes that this latter experiment can easily be made on a large scale, and that if Mr. Parsons had gotten a mortar contact between the bottom of his can and the sand, and had allowed time for setting up and adjustment, his results would have been different. He believes also that the best way to make the uplift experiment on a large scale is to use a large piston in a hydraulic chamber and plunge the piston deeply into the sand, giving it time for settlement and adjustment before putting on pressure; that is, repeating the hydraulic chamber experiments made by the writer* with a much larger piston and apparatus.

In each case, care was taken to get contact and, at the same time, to see that the cap or piston was as free to turn as in the case of the clear water trial.

It is probable, however, that full contact cannot be established in small areas such as this; but this experiment is believed to prove definitely that the uplift area is greatly diminished by the presence of sand, gravel, or soil of any kind.

The writer is now practically convinced (*a*) that the area of uplift pressure on a submerged structure, buried or partly buried in soil, is reduced below the percentage of voids rather than equalling it, or tending toward 100% of the whole area; (*b*) that the pressure of water is against a mass of soil rather than through it, unless leads, orifices, or artificial voids occur; and then (*c*), unless such leads or artificial voids are made to exist, that the flow of water through soil is rather by seepage than by pressure head and, although this head influences the flow, the influence is incidental rather than direct, and in proportion to the void characteristics of the soil.

Daugherty states:†

"Thus in the case of a ship, for example, sunk in the mud at the bottom of a body of water, the pull, *T*, necessary to raise the ship is not only the weight of the ship, but also the entire volume of water on top of it."

This is probably true of all structures buried or partly buried in submerged soils which do not contain a large percentage of voids natural or artificial.

The speaker believes that pressure is rapidly dissipated when it has no outlet, just as it has been found impossible to force grout or pea gravel under pressure into natural voids; but when it has an outlet, grout soon opens up a lead and can be forced into artificial voids long distances away from the pump.

Here it may be interpolated that rust, too, will fill artificial voids rapidly, but when these are filled it will cease to function, except as a protective coating for the remaining steel or iron.

Returning to the matter of pressure through voids, the speaker, through his assistants, continued the experiments with the apparatus shown in Fig. 7(*a*) and found that when gravel was placed in the bottom of the chamber and sand to a depth of 4 in. was pressed down compactly above it with a contact cap of grease between the sand and the piston cap, the sand was

* *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 364.

† "Hydraulics", p. 33.

forced out of the chamber under a column equal in head to from 1.8 to 2.1 ft. This finding showed two things definitely: It took from 75 to 100% more head of water acting through gravel to raise the sand column from a gravel contact than was theoretically required to lift its own weight, together with that of the cap. That is, the weight of the cap and sand together was 2.6 + 2.8 = 5.4 lb., which, distributed over 12.56 in. of area, equalled 0.43 lb. per sq. in., or approximately a column of water of 1 ft., which would indicate a contact of from 40 to 50% between the sand and gravel. It further showed that the line of least resistance was not through the voids to lift the lighter cap which would raise in clear water at 6½ in. of column, but rather against the heavy body of sand requiring a column of from 1.8 to more than 2 ft. This, and a previous experiment,* proves definitely that where there are no artificial voids, uplift pressure is greatly diminished or absent on partly buried submerged structures.

The history and record of such structures are full of such evidence. Among typical examples may be cited: (*a*) A structure concreted to solid rock in which there are no fissures and, consequently, no uplift; (*b*) dredges flooded and sunk which do not rise when pumped out and have to be "hydraulically mined" to raise them; (*c*) hulks half buried in sand which cannot be raised through buoyancy alone; (*d*) tunnels and structures which do not rise after the soil is stabilized around them, although difficulties are sometimes experienced in holding them down during the exigencies of construction; and numberless others. This is definite evidence that theory and practice must here be in accord. Sometimes the instances cited are noted as due to suction, but there can, of course, be no suction unless the pressure is almost or entirely excluded from the otherwise buoyant area.

It is generally known that the Holland Tunnel rose more than 12 in. due to the hydraulic envelope caused during construction by the agitation into a soupy condition of the surrounding soil, and that it had to be partly backfilled to prevent flotation; and yet it cannot be doubted that when this hydraulic envelope had been replaced by grout and the surrounding soil stabilized, that there could no longer be any possibility of the tunnel rising.

Light back-fill, or friction grip, even if it may be more considerable than found by the author,† will not hold down a structure which has a water-jacket, or is under what may be called full buoyant pressure. The writer agrees with the author that it is always safest to design, where practicable, against full pressure area, particularly in structures subject to heavy vibration, shallow submergence, or shallow cover. This is equivalent, however, to saying that steel structure designs should have a factor of safety; but it does not militate against the theory that there cannot be pressure and contact in the same area. The writer wishes to thank F. L. Cranford and C. H. Locher, Members, Am. Soc. C. E., and their staff for assistance in making the experiments noted.

* A photograph of the sand column lifted in a similar experiment made by the speaker is shown in *Transactions*, Am. Soc. C. E., Vol. LXX (1910), p. 364, Fig. 22.

† *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 945.

EUGENE E. HALMOS,* M. AM. SOC. C. E.—The speaker had the privilege to be in constant touch with the work done by the author in devising and making the tests and in the interpretation of the results. He believes that the method of approaching the problem and the manner of conducting the tests will be found to be above criticism and that the author's conclusions will meet with the approval of the profession.

What Mr. Parsons did not state but of which the speaker knows, is the large amount of labor and the considerable time the author has devoted to these tests and the substantial sums of money which he has expended in order to find the answer to a question the solution of which he considered of commensurate importance for the benefit of the Engineering Profession. Moreover, there was the agony of suspense lasting for days and often for weeks at a time when, due obviously to errors in technique somewhere, results contrary to reasonable expectation were recorded. These errors could not be easily detected on account of the complexity of the elements which, by the nature of the problem itself, go to make up the results of the tests. The whole apparatus and every step of the experiment had to be re-examined with painstaking care many times before all sources of error were eliminated.

Most of the experiments were conducted at Stevens Institute and at Rensselaer Polytechnic Institute. It was found that the engineering colleges, perhaps with very few exceptions, are not properly equipped to make tests of a variety of practical problems in their hydraulic laboratories. In the case under discussion, the efforts and the money of a private individual had to be utilized in the solution of a problem which is of interest to all engineers. The speaker believes that this is another instance that indicates the necessity of installing modern hydraulic laboratories in American institutions of learning.

As to the paper itself, the speaker wishes to comment on one point only which has not been emphasized by the author, probably because it is somewhat of a by-product.

In making the tests to determine the friction on the sides of the cylinders, the author has found a formula which shows that the average friction per unit of surface is proportional to the 0.4 power of the depth to which the cylinders were embedded. The total friction per linear unit of perimeter, therefore, would be proportional to the 1.4 power of the depth. It is an accepted principle in mechanics that within the limits of the pressures here considered, the friction is proportional to the pressure exerted against the surface. Therefore, the pressure of the sand around the cylinder has been shown to be proportional to about the 1.4 power of the depth, contrary to the formulas used in the design of retaining walls, dock walls, etc., which assume that pressure of both dry and saturated earth-fill is proportional to the square of the height of the backing. In other words, the author's formula gives a much smaller pressure than that of Rankine. It seems that the difference, which is very substantial, is due to cohesional forces acting between the particles of sand used for the tests. These forces are neglected in the formulas derived to determine the pressure on retaining walls. On the other hand,

* Chf. Engr., Parklap Constr. Corporation, New York, N. Y.

when sinking caissons in saturated ground to substantial depths, practical constructors assume uniform friction along the outer surface of the caisson, the value of which they find to be independent of the depth, in other words, proportional to the zero power of the depth. The author's formula, therefore, gives values greatly in excess of the friction experienced in such cases.

In view of the foregoing, the speaker believes that it is well to emphasize the fact that the formula given by the author, while correct within the limits of his experiments and for the material utilized in his tests, should not be used for the purpose of designing retaining walls, caissons, or similar structures.

T. R. LAWSON,* M. AM. SOC. C. E.—Emphasis should be placed on the care taken by the author to eliminate all possible variables and to secure conditions which will approximate those of actual practice. The 12-in. plunger used in the apparatus at Rensselaer Polytechnic Institute was a hollow cylinder filled with concrete, thereby giving a contact between a concrete base and the sand, thus simulating the actual conditions under the base of a concrete dam.

In order to prevent any trouble from settlement, no readings were taken until after the full load had been applied through the concrete weights, and a sufficient interval of time had been allowed between loading and readings until all possibility of settlement had ceased. This time interval ran from 24 to 48 hours.

When these experiments were first begun, some difficulty was experienced from the expansion of the large cylinder from internal pressure. The difficulty was made apparent by observing that, under pressure, the piston settled into the sand instead of rising. The difficulty was overcome by heavily reinforcing the outside container with steel bands. No trouble was experienced after this up to pressures of 30 lb. per sq. in.

The series of experiments were repeated a number of times under different loads on the piston, and in each instance the upward motion of the piston began at a load approximately equal to the friction of the sand on the side of the piston plus the product of the area of the piston and the pressure per square inch indicated by the pressure gauge.

This apparatus is still available for further tests, and should it appear desirable as a result of the discussion of this paper, other experiments will be attempted.

If the upward pressure on a pervious soil is 100%, as indicated by the experiments, considerable attention must be paid to cut-off walls not only on the heel to take care of the seepage under the base, but also at the toe where the full static pressure from the tail-bay will act on the base.

The results from these experiments bring forcibly to mind another very important research on which there is but little reliable data; that is, the thrust due to a saturated fill on the back of a retaining wall. While retaining walls are designed with a fairly accurate knowledge as to the thrust due to a dry fill, very little information is available to the engineer as to the effect of sat-

* Head, Dept. of Civ. Eng. Rensselaer Polytechnic Inst., Troy, N. Y.

urating this fill with water. Information of this character would be particularly useful in the design of harbor and dock walls where the effect of tide on the back of the wall is an important factor.

J. VIPOND DAVIES,* M. AM. SOC. C. E.—The speaker notes with interest that engineering investigators differ on the matter of hydrostatic uplift in soil. When he first came in touch with the question of the construction of tunnels under the Hudson River, he was told that there were two schools of engineering thought on the subject. One was that any tunnel built under the Hudson must of necessity float up to the surface by uplift, and then float down the river. Members of the other school insisted that it was absolutely certain that any tunnel built under the river would sink in the comparatively soft silt and that would be the end of it.

So far as the speaker knows, every tunnel built under the Hudson River has, at some time or other during its construction period, been weighted to keep it from lifting. The Hudson and Manhattan Tunnels were all weighted by the excavated silt. This was left on the floor of the tunnel until such time as the superincumbent load or flow of the soil, comprising the river silt, squeezed in on the tunnel lining and gave weight enough to resist the buoyancy of the tunnel itself. Subsequently, the tendency was toward a downward settlement notwithstanding that the tunnels are much less in weight than the displaced soil.

It is a fact that any dredging in the bed of the river causes uplift in the tunnel below. There is no doubt about that; it is a perfectly measurable quantity. During the recent dredging of the 2 000-ft. channel in the Hudson River to a 40-ft. depth plane, there was a very definite uplift in all the tunnels under the river, caused by the removal of that superincumbent load.

Many engineers are familiar with the fact that all tunnels in Hudson River silt pulsate with the daily rise and fall of the tide, sinking as the tide rises and the pressure on the silt bed increases, and lifting as the tide falls and the pressure on the bed of the river reduces.

It is equally certain that any tunnel under the Hudson can be lifted bodily by pumping stiff puddled clay, or some material like clay, through the invert. The tunnels normally remain stable and stationary under the river; but there is no doubt that a tunnel can be lifted if that were desired for any reason. It can be raised and uplifted by high-pressure pumping of a solid material.

As to the distribution of the pressures over areas, the speaker has never had the assurance to do otherwise than consider 100% of the hydrostatic uplift pressure uniformly distributed over the entire area of structures that were likely to be in contact with water.

As an example, in the Hoboken Terminal of the Hudson Tunnel System, a station structure was designed as a box with complete arched inverts, partly in silt and partly in sand. In that case the entire structure was designed to give 100% loading against hydrostatic uplift. There were parts in which the weight of the structure itself would have been less than the aggregate

* Cons. Engr.; Pres., Jacobs & Davies, Inc., New York, N. Y.

load necessary; and in those sections the speaker had the space under the platforms filled with concrete to increase the loading against uplift. These localized areas were (*a*) at the east end of the station where the silt was very soft, being near the water-front; and (*b*) where the concourse floor above the main structure reduced the amount of back-fill forming part of the superincumbent load.

Any man would be quite brave who attempted to design structures of this character, without allowing fully for the entire hydrostatic uplift, uniformly distributed over the entire area.

LAZARUS WHITE,* M. AM. SOC. C. E.—As a result of experiments performed in 1910,† the speaker was rather inclined toward the belief that there was not 100% hydrostatic uplift on structures built in pervious soils. Mr. Parsons' tests were made with the expectation of rather confirming the same views and when the results were found to be inconsistent with his ideas they were studied very carefully. The speaker has been unable to find flaws in these later experiments.

The question arises, How can one reconcile the older tests with those made by the author? They were both honestly made and they are radically different in their conclusions. The speaker believes that the latter were conducted under truly static conditions and the former under dynamic conditions. In the older experiments there was a flow of water in the sand and a big loss of pressure.

As Spencer has said "there is some truth left in every error". However, because there is an uplift on 100% of the area it does not follow that there is 100% pressure, except in the case of a truly static condition, which does not usually exist in construction work. It is, therefore, necessary to multiply a 100% area by a pressure which is less than the full hydrostatic pressure.

Suppose, for instance, that there is 100% on a concrete floor in a coffer-dam. Through leakage there is a big reduction of head, so that the total uplift is much less than the maximum which can be computed. For that reason those who have constructed coffer-dams incline to the belief that uplift does not act on the full area. The true explanation is that it is acting on the full area, but not with an intensity corresponding to 100% of the difference of head between the inside and outside water levels.

It is well known that 20 ft. of head can be supported by 1 ft. of concrete, provided the foundation material is well under-drained and the leakage is pumped away. Of the concrete, 1 ft. will suffice because allowing that flow gives a tremendous loss of head between inside and outside. In quite a number of cases the need of allowing for 100% uplift has been obviated by permanent under-drains and pumps. This is a natural condition as much as anything else of which the engineer may take advantage if he sees economy in it.

T. KENNARD THOMSON,‡ M. AM. SOC. C. E.—It seems to the speaker that when subsoil materials are taken into a laboratory they are no longer sub-

* Pres., Spencer, White & Prentis, Inc., New York, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXX (December, 1910), p. 352.

‡ Cons. Engr., New York, N. Y.

soil materials. They can never again be put into a condition duplicating that in which Nature had placed them or giving similar test results. For instance, in each and every case there is a difference in the arrangement of the more or less natural cementing material, and there is no way of telling how many thousands of pounds pressure Nature may have exerted during the placing or shortly thereafter.

In Western New York State the speaker has seen hardpan, or glacial drift, that in some former age had been placed under an unknown depth of water, at an elevation of more than 1 200 ft. above sea level. One might almost as well pulverize concrete and then expect it to act in a laboratory as if it still were concrete.

An accident in one of the Hudson River tunnels would be appalling. However, the fact remains that all the Hudson River tunnels are lighter than water and are subject to slight daily variations in level-rising and falling inversely as the tide, although, of course, the theoretical hydrostatic uplift should depend entirely on the difference in elevation between the top and bottom of the tunnel and not on the total depth of the water.

What probably happens is that the effect of the tide re-acts more quickly on the roof than on the bottom of the tunnel, allowing the water to be squeezed away from the base, so that when the tide rises the tunnel is forced down into the silt, reversing the process as the tide falls. While no one can prove that they will ever be wrecked, no one can prove that they will not. It would be worse than foolish to make this statement—except as a warning against constructing future Hudson River tunnels on the same basis as the present tubes. Beyond a peradventure of a doubt, perfectly safe tunnels can be built, and for much less money, in the Hudson River.

A circular caisson, carried to hardpan in Lower New York, with concrete 4 ft. above the cutting-edge in the working chamber, was lifted by the water pressure and had to be removed, at a considerable loss. On another occasion a large rectangular caisson resting on rock—at a depth of 19 ft. of water in the Susquehanna River, with the 6-ft. working chamber filled with concrete and concrete also placed above the deck—was lifted by an unusually high tide and had to be towed away and destroyed. In that case if there had been 1 ft. more of concrete on the deck, the mass would have been heavier than the water displaced. Each and every foundation is apt to have certain conditions not found elsewhere and all the laboratory tests in the world would not eliminate the necessity of using one's best judgment. It would appear, therefore, that the only safe rule for uplift is to use 100% of the hydrostatic head.

DANIEL E. MORAN,* M. AM. SOC. C. E.—The speaker would like to emphasize two points. When water is moving through sand there are static conditions and dynamometric conditions, and they must not be confused. Moreover, there is capillary action. If the base of a layer of fine sand is wet, the water will rise in the sand by capillary action. In laboratory experiments on sand there are radically different conditions than those found in a natural deposit.

* (Moran, Maurice & Proctor), New York, N. Y.

The second point is that in making various tests in the laboratory it became very evident to the speaker that if he loaded sand in a cylinder, the forces transmitted from the loaded piston to the platen went partly through the sand and partly through the cylinder. He then substituted for the platen two pistons, one on the top and one on the bottom, with only a relatively small volume of sand in the cylinder.

When the speaker applied the load on the top piston and tried to measure it on the bottom piston, part of the load was being transmitted through the sand and part through the walls of the cylinder containing the sand. Therefore, he had to be very careful in reaching any conclusions.

The author's experiments were undoubtedly accurate and designed to secure accurate results. The only mystery to the speaker is that he did not get 100 per cent.

ALEXANDER V. GALLOGLY,* ASSOC. M. AM. SOC. C. E.—Some years ago the speaker was engaged in the construction of an earthen dam. The reservoir was formed by cutting off the top of a hill and depositing the excavated material around the edge to form an enclosure about ¼ mile wide and ½ mile long The excavated material was ideal, consisting of about two-thirds sand and one-third clay and boulders.

The dam was built in 8-in. layers and rolled with a 10-ton grooved roller. No stones larger than 8 in. in thickness were included. The resulting material had nearly the nature of concrete. This part of the dam (which formed the inner portion) was called the "impervious embankment", and it proved to be quite impervious. It was about 180 ft. thick at the base, 30 ft. at the top, and it had a maximum height of 42 ft. Excavated material, including boulders, was placed around the outside in 2-ft. layers without rolling.

About four years after the reservoir had been filled with water, the work of cutting through the dam for the purpose of tapping the lake was undertaken. The material was found to be dry and acted like concrete, there being no curve of leakage, such as that described in textbooks. This condition continued until the original ground surface was reached and passed. When the bottom of the trench had been excavated to a point about 1 ft. below the original surface, a small pool of water about 2 ft. in diameter appeared. This was bailed out, and the material was excavated with a shovel until the water began coming through the fine sand lenses in a manner similar to that observed in the excavations outside the reservoir. The level at which this water appeared was 21.5 ft. below the surface of the water in the lake. As the excavation proceeded, this condition gradually developed over the entire area of the bottom and sides of the trench below the level of the original surface under the dam.

As the trench was being excavated and just before the water appeared, a peculiar hissing sound was heard. This was found to be caused by air which was being forced out ahead of the water. If upward pressure was present to any extent, why did it not push up the soil in the bottom of the trench? Did not the presence of the trapped air cause an additional weight to act

* Care, Carpenter & Duffy, Mamaroneck, N. Y.

against the hydrostatic head? During the construction of the gallery, the seepage into the trench amounted to only 300 gal. in 24 hours. This was removed by bailing.

The speaker believes that the conditions surrounding a foundation placed in water-bearing material, can be compared to a U-tube closed at one end, in which the water in the open leg is balanced by the foundation itself, together with that portion of the material under the foundation in which air is trapped. It is known that all water contains small air-bubbles which tend to collect under a flat surface.

In the experiments described by the author, the apparatus seems to illustrate the conditions surrounding a river caisson only. In such a case the upward pressure would equal the full hydrostatic head plus the upward pressure caused by the weight of the semi-liquid material penetrated by the caisson. The experiments do not seem to cover all the cases of foundations constructed in pervious materials below ground-water level.

ALLEN HAZEN,* M. AM. SOC. C. E. (by letter).†—This paper presents a careful experimental investigation of one of the fundamental conditions of dam construction. It is no less valuable because it affords a complete confirmation of ideas long held by many members of the Society. It should lead to the discard of all less complete allowances.

The effect of uplift from water pressure is not confined to material below the base of masonry dams. It may also exist in any part of the structure where the masonry is pervious unless adequate interior drainage is provided. In cold climates frost may cause a temporary sealing of outlets at the lower face of the masonry and this, in turn, may result in development of full pressure in the full thickness of a dam.

The effect of uplift is to reduce the stability and strength of the dam. It tends first to overturn it and, second, to take off weight and reduce the friction and resistance to sliding.

The resistance of natural materials to sliding is not known with any satisfactory degree of definiteness. There is reason to think that with some softer rocks, the resistance to sliding is not in proportion to height and weight; and for high structures it may be much less than has sometimes been assumed. The Panama Canal slides furnish an illustration, from an entirely different field, of the low coefficient of friction of soft rocks at high pressures. Experience with harder materials and lower unit stresses may not be a safe guide for soft rock. Masonry dams have failed by sliding more frequently than by overturning.

Looking at the matter broadly, the problem of dam design needs to be managed with a strong hand. Dams and reservoirs must play an important part in the future development of this country. They must be made strong enough and safe enough so that the people who live near them as well as those who build them and benefit by them will be assured of their safety.

In the past, some dams have been built of ample proportions strong enough for all contingencies. There are many dams in the United States that would

* Cons. Engr. (Hazen & Whipple), New York, N. Y.

† Received by the Secretary, May 1, 1928.

stand if filled with a liquid twice as heavy as water. These dams are safe. Even earthquakes do not seriously shake or touch them. On the other hand, there are many dams that would not stand such a test. Too many dams have failed to reach the point of actual stability.

Engineers speak of a factor of safety, but the term needs to be defined more closely. In a steel structure the factor of safety may mean the ratio between the stresses at the assumed loading and the ultimate strength of the steel, but no one thinks that a load corresponding to the ultimate stress could be safely applied. Engineers speak of a factor of safety of 4, but really they believe that something like double the assumed load would probably be held.

It may be somewhat the same with dams. When a designer states that a factor of safety of 2 was used—based no doubt on some rather arbitrary assumptions—it may mean that the real factor of safety is much less than 2. It may turn out that it is little, if any, more than 1. It certainly does not mean that a liquid twice as heavy as water could be safely held.

It may be earnestly hoped that some one will set up an authoritative policy of dam design that will lead to the construction of dams that would be stable and safe if subjected to pressure from a liquid much heavier than water. If this could be done, it would tend to restore confidence in dam design and construction; and even if some of the works to be presently constructed, should cost somewhat more than they otherwise would, some such policy is needed and will tend to the ultimate benefit of all concerned.

CHARLES TERZAGHI,* M. AM. SOC. C. E. (by letter).†—The hydrostatic uplift acting through sands on the bottoms of seals, cellar floors, and other structures, is so simple and clear a hydraulic phenomenon that it is far more difficult to account for the attitude of engineers toward the uplift than it is to compute its intensity.

The theoretical aspects of hydrostatic uplift acting through sands were convincingly presented in 1912 by A. Franzius,‡ and it is almost inconceivable that the opponents of Mr. Franzius§ could fail to realize the weight of his arguments. Gaede, in 1917,‖ and Engesser, in 1919,¶ computed the intensity of the uplift acting through sand, assuming that the area of contact between the individual sand grains is determined by the Hertz formula (area of contact between curved surfaces pressed against each other). The computation led to the conclusion that the hydrostatic uplift acting on a surface in contact with the sand should range between 97% and almost 100% of the hydrostatic uplift exerted by water, provided the modulus of elasticity of the sand grains ranges between 100 000 and 500 000 kg. per sq. cm., and that the pressure acting on the sand does not exceed 10 tons per sq. ft. The fraction of the area over which the hydrostatic uplift does not act should, according to theory, be independent of the size of the grains. These theoretical results

* Associate Prof., Foundation Eng., Mass. Inst. Tech., Cambridge, Mass.

† Received by the Secretary, May 3, 1928.

‡ *Zentralblatt der Bauverwaltung*, 1912, p. 583; 1913, p. 617.

§ *Loc. cit.*, 1912, pp. 522 and 617; and 1913.

‖ *Loc. cit.*, 1917, p. 501.

¶ *Loc. cit.*, 1919, p. 443.

agreed with the results of the experiments performed in 1886 by Brennecke,* in 1887, by Forchheimer,† in 1916, by Schaper,‡ and in 1917, by Busemann.§ They also are in very good agreement with the results of the tests performed in 1926 by Mr. Tsin Hsiao, at the Massachusetts Institute of Technology, under the supervision of the writer, and in moderately satisfactory agreement with those now published by the author.

All the tests, including those of Mr. Parsons, were based on the same principle, namely, measuring the hydrostatic uplift at the instant when the forces acting on the submerged body balanced those acting downward. Because of this fact, Busemann claimed in 1917 that the test results thus obtained do not represent the hydrostatic uplift which acts on the base of a pier before an actual uplifting occurs. However, his arguments can be considered conclusively invalidated by the results of Gaede and the computations of Engesser.

As a matter of fact, the data and assumptions on which the theory is based, are so plain and the range of possible error associated with theory is so small (if compared with the importance of the errors which are apt to be connected with laboratory tests in that particular field), that in all cases where the test results are in conspicuous contradiction with theory, investigators should first direct their attention toward finding errors in the testing method. It should be kept in mind that a real contradiction between test results and theory—that is, a contradiction which cannot be accounted for by errors of observation—would mean that there is an error in the fundamental principles of hydrostatics, and there are not many sciences which rest on so stable a foundation.

There is such a contradiction, for instance, between the results of the Engesser theory and the conclusions reached by Mr. Parsons concerning the effect of the depth of the permeable stratum on the intensity of the hydrostatic uplift. According to theory, the thickness of the layer of sand between the base of this structure and the impermeable bottom below the sand should be without any effect on the hydrostatic uplift, provided it is at least equal to several times the diameter of a sand grain. According to Mr. Parsons, the uplift is reduced when the depth of the pervious soil is shallow. The writer believes that the apparent influence of the thickness of the layer was due to the air content of the sand beneath the base of the can. Undoubtedly, Mr. Parsons has done his best to exclude this source of error, but every one who has had experience with sand experiments knows that it is almost impossible to exclude the air entirely. Thus, for instance, when saturating a sand by allowing the water to enter through a permeable base, thus giving the air a chance to escape freely in an upward direction, it was found by weighing and by computation that, within the saturated space, more than 30% of the voids remained filled with air and the degree of saturation increased very slowly as time passed. If it has no possibility of escaping in an upward direction because of the presence of the bottom of the can, it is hardly to be

* *Zeitschrift für Bauwesen*, 1886, p. 101.

† *Zentralblatt der Bauverwaltung*, 1887, p. 314.

‡ *Loc. cit.*, 1916, p. 514.

§ *Loc. cit.*, 1917, p. 205.

expected that the air will ever disappear completely. One of the possible effects of the air content on the hydrostatic uplift, if measured by high-head tests, can be explained by Fig. 8. The dotted line indicates the size of an air bubble at the outset of the test. As soon as the water pressure goes up, the bubble becomes smaller and this, in turn, requires the water to flow toward the bubble. Then the head required to overcome the resistance against flow causes an apparent increase in the pressure required to lift the can, because the true resistance consists of both the weight of the can and the head required to direct the flow toward the shrinking bubbles.

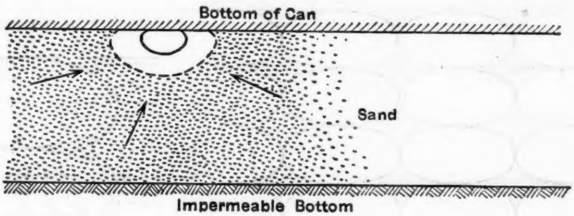

FIG. 8.

In the tests which were performed in 1926 by Mr. Tsin Hsiao, at the Massachusetts Institute of Technology, the diameter of the can was 5 in. In order to reduce the effect of any trace of air on the test results, the sand was placed on top of a layer of coarse sand (grain size, from 2 mm. to 6 mm.), so that the distance between the bottom of the can and the top of the coarse sand was nearly equal to 1 in. The results of the tests were as follows:

Grain size, in millimeters.....	2.00	1.41	0.84	0.35	0.10
Effective area, in percentage....	99.25	99.85	99.50	98.50	99.25

Considering these figures and the fact that the results obtained by Mr. Parsons are as far from being as consistent with each other as those obtained by Brennecke in 1886 and by Forchheimer in 1887, the writer hesitates to accept his conclusions concerning the effect of the thickness of the layer on the intensity of the hydrostatic uplift. The writer is inclined to consider this effect exclusively as a result of various imperfections of the testing method. However, as a whole, Mr. Parsons' tests furnish figures that are at least of the same order of magnitude as those which ought to be obtained according to the laws of hydrostatics. Hence, it is hoped that they will serve their purpose of demonstrating to the skeptic members of the Engineering Profession (for the sixth time since 1886) that the uplift acting through sand really is very nearly 100% of the uplift acting through water, provided the underground consists of almost cohesionless sand. If Mr. Parsons succeeds in achieving this purpose, his efforts will certainly have been well worth while.

Far more difficult is the question concerning the hydrostatic uplift acting through fine silt and clay. For measuring the uplift in such materials, no satisfactory experimental method has been found thus far—Mr. Parsons' method included. The theory of Engesser also loses its validity because every soil grain is surrounded with a layer of adsorbed water which has, for all intents and purposes, the properties of a solid. No hydrostatic uplift can

be transmitted through this layer, the thickness of which is certainly not more than $\frac{1}{20\ 000}$ mm.; it may be thinner. For sand grains, the presence of this layer can be neglected. Hence, for such materials, Engesser's theory is accurate enough. However, for grains the size of very fine silt and clay particles, these layers are apt to become an item of increasing importance.

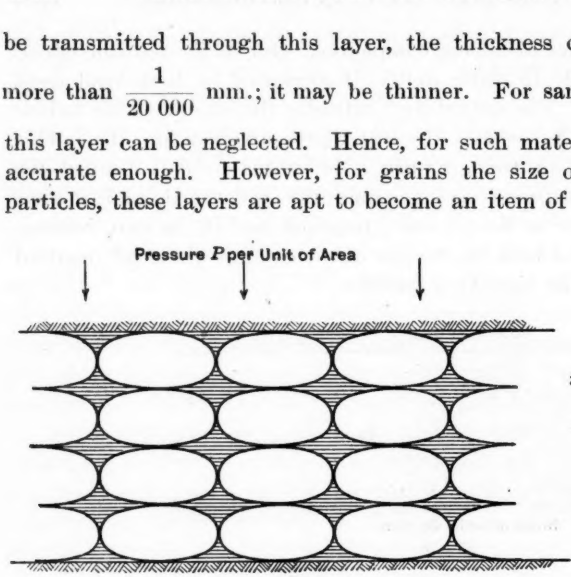

Fig. 9.

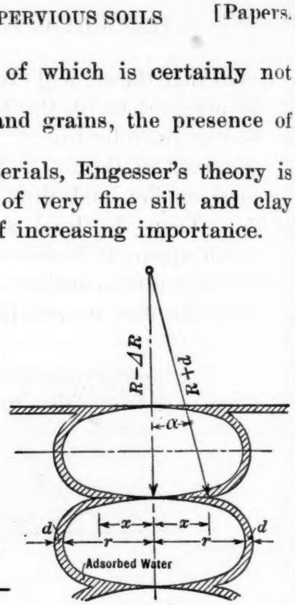

Fig. 10.

For evaluating the influence of the layers of adsorbed water on the hydrostatic uplift, assume that the bottom of a can is supported by lens-shaped grains arranged as shown in Fig. 9. The grains touch each other at their flat sides. Each grain is surrounded with a skin of adsorbed water with a thickness, d (Fig. 10). Since, at the bottom of the can, the area exposed to the uplift is smaller than that between the grains, the computation must be made for a section beneath the first row of grains (Fig. 10).

Let E = the modulus of elasticity of the grains;
r = the radius of their vertical projection;
n = the shape factor;
$R = n\,r$ = the radius of curvature of their surfaces of contact;
P = the pressure per point of contact;
p = the downward pressure exerted by the can on the sand (including hydrostatic uplift);
x = the radius of the area over which no hydrostatic uplift can act;
F = the proportion of a unit of area of the base upon which no hydrostatic uplift can act; and,
$\varDelta R$ = the shortening of the radius of curvature, R, due to the pressure, p.

The number of points of contact per unit of area of the base is $\frac{1}{4\,r^2}$. Since the angle, α, is very small, the radius, x, is almost equal to,

$$2\,R\,(d + \varDelta R) = 2\,n\,r\,(d + \varDelta R)$$

For R, the formulas of Hertz give the term:

$$\varDelta R = 0.77\sqrt[3]{\frac{P^2}{E^2\,R}} = 1.94\,r\sqrt[3]{\frac{p^2}{n\,E^2}}$$

The area over which the hydrostatic uplift cannot act is,

$$F = \frac{x^2}{4\,r^2} = \frac{1.57\,n}{r}\left(d + 1.94\,r\sqrt[3]{\frac{p^2}{n\,E^2}}\right) = 1.57\left(\frac{d\,n}{r} + 3.05\sqrt[3]{\frac{n^2\,p^2}{E^2}}\right)$$
$$= 1.57\,(C_1 + C_2) \dots\dots\dots\dots\dots\dots\dots\dots (1)$$

in which, $C_1 = \frac{d\,n}{r}$ and $C_2 = 3.05\sqrt[3]{\frac{n^2\,p^2}{E^2}}$.

From Equation (1) the value, F, is found to be independent of the grain size, provided the quantity, $\frac{d\,n}{r}$, is so small that it can be neglected. In addition, this formula gives a conception of what the values of F could approximately be, assuming that d is equal to or smaller than $\frac{1}{20\,000}$ mm. = 0.000005 cm. Sand grains touch each other at some points of contact along flat surfaces; at others, there are corners resting against faces. For average conditions, $R = r$, or $n = 1$, but for clays (due to the abundance of scale-like particles) the designer may assume tentatively that $R = 5\,r$, or $n = 5$.

If these values are introduced into Equation (1), the results obtained are as shown in Table 8.

TABLE 8.

		Bulky particles, $n = 1$.	Scale-like particles, $n = 5$.
(*A*) Grain Size, 0.2 Mm. (r = 0.01 Cm.)			
p = 0	C_1	0.000500	0.00250
	C_2	0	0
	F	0.000784	0.003930
Effective area, in percentage	....	99.9	99.6
p = 5 kg. per sq. cm.	C_1	0.000500	0.00250
	C_2	0.001424	0.00416
	F	0.003024	0.0104
Effective area, in percentage	....	99.7	99.0
(*B*) Grain Size, 0.002 Mm (Limit Between Silt and Clay, r = 0.0001 Cm.)			
p = 0	C_1	0.05000	0.2500
	C_2	0	0
	F	0.0784	0.3930
Effective area, in percentage	....	92.2	60.7
p = 5 kg. per sq. cm.	C_1	0.05000	0.25000
	C_2	0.00142	0.00416
	F	0.081	0.398
Effective area, in percentage	....	91.9	60.2

The figures assembled in Table 8 (A) show that there is no possibility of accounting for an effect of the presence of sand or gravel on the hydrostatic uplift in excess of about 1%, provided the laws of hydrostatics are valid. The differences between 0.99 and Mr. Parsons' figures can safely be considered a measure for the efficiency of his testing arrangement.

In connection with the figures quoted in Table 8 (B) it should be remembered that the films of adsorbed water which exclude the transmission of hydrostatic uplift, also produce an adhesive bond between the particles. In other words, the greater the pressure-reducing effect of the films, the greater should be the cohesion of the material. This conclusion agrees with the empirical fact that a microscopic sand ($n = 1$, with small values of F) has very little cohesion if compared with a clay ($n = 5$, with large values of F). Hence, the greater its cohesion, the greater should be the pressure-relieving effect of the presence of the substratum on the hydrostatic uplift. Absence of cohesion is a safe indication for almost 100% uplift.

In connection with the sealing of foundation pits, prior to unwatering the sealed space, there is still another factor which requires serious consideration. It is the relative permeability of the seal.

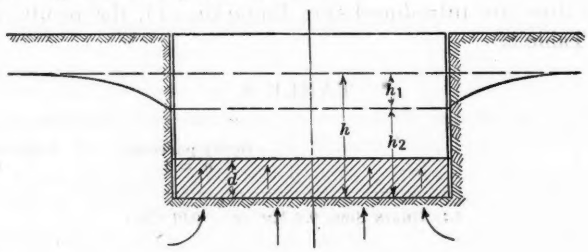

Fig. 11.

Fig. 11 represents a cross-section through a round or a square foundation pit surrounded by a water-tight enclosure and sealed by a somewhat permeable bottom slab with a thickness, d.

Let F = the area of the bottom of the pit;
k_1 = the coefficient of permeability of the seal;
k_2 = the coefficient of permeability of the underground;
h = the hydrostatic head which would act on a perfectly impermeable seal; and,
h_1 = the loss of head due to percolation through the seal.

According to the law of Darcy the quantity of water, Q, which percolates through the seal is equal to,

$$Q = F\, k_1 \frac{h_2}{d}$$

On the other hand, the loss of head, h_1, due to the quantity, Q, flowing through the underground toward the seal is (according to Forchheimer) approximately equal to,

$$h_1 = \frac{Q}{4\, k_2 \sqrt{\frac{F}{\pi}}}$$

Since $h_1 + h_2 = h$,

$$F\,k_1\,\frac{h - h_1}{d} = 4\,k_2\,h_1\sqrt{\frac{F}{\pi}}$$

The relative loss of head, $\frac{h_1}{h}$, that is, the apparent decrease of the hydrostatic uplift due to percolation, is therefore, equal to,

$$\frac{h_1}{h} = \frac{1}{2.26\,d\,\frac{k_2}{k_1}\,\frac{1}{\sqrt{F}} + 1}$$

Suppose that the seal is ten times less permeable than the underground; that is, $k_1 = 0.1\,k_2$; that F_1, the area of the pit, is 100 sq. m. (about 1 100 sq. ft.); and $d = 1$ m. (3.3 ft.). Under these conditions, the drop in head due to percolation would be,

$$\frac{h_1}{h} = \frac{1}{3.26} = 0.307 = 30.7\%$$

For $k_1 = 0.01\,k_2$,

$$\frac{h_1}{h} = \frac{1}{23.6} = 0.0425 = 4.25\%$$

provided the underground is not stratified and that it is equally permeable to a great depth. In practice the underground almost always contains layers and seams of silty material that are only slightly permeable, and the corresponding drop of head would be much more important.

From all these considerations, the writer draws the following conclusions: The difference between the hydrostatic uplifts measured by Mr. Parsons on the bottom of the cans and the hydrostatic uplift exerted by plain water may essentially be due to various imperfections of the testing method. The same seems to be true for the apparent effect of the depth of the pervious soil on the intensity of the hydrostatic uplift. The apparent contradiction between theory, according to which practically 100% of the hydrostatic uplift should act through cohesionless materials, and engineering practice which shows that the uplift through such materials is sometimes very much smaller, may essentially be due to the pressure-relieving effect of seepage through the seal, seepage through joints in the enclosure, and the adhesion between the seal and the enclosure of the pit. On the other hand, if the underground has an appreciable cohesion, the low value of the uplift may be due, wholly or partly, to the physical character of the water enclosed in very narrow spaces.

Knowing by experience the great amount of time and painstaking labor required to perform an investigation of the type described by Mr. Parsons, the writer wishes to express his appreciation, and it is hoped that the paper will serve its purpose of clearing up the prevailing misconceptions concerning hydrostatic uplift.

FRANK S. BAILEY,* ASSOC. M. AM. SOC. C. E. (by letter).†—The author's researches have provided a welcome addition to the fund of engineering

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† Received by the Secretary, May 25, 1928.

knowledge. The experiments were repeated so many times and the results interpreted so carefully that there seems to be no question as to the reliability of the conclusions. They should bring to an end the uncertainties and differences of opinion which may have hitherto existed.

Now that Mr. Parsons' experiments have been published, it is to be expected that records of similar experiments or of measurements of uplift pressures on various structures will be added. By coincidence some records of this nature have appeared,* describing uplift measurements made by the U. S. Bureau of Reclamation on several dams. Two of these records—the Colorado River Dam, on the Grand Valley Project, in Colorado, and the Percha Dam, on the Rio Grande Project in New Mexico—show upward pressures under dams which are founded on gravel.

Records of uplift pressures on the Pinhook Dam, at Maquoketa, Iowa, and on the Island Park Dam at Dayton, Ohio, show that both dams rest on sand and gravel foundations.† These experiments showed that, unless the bed of the stream or reservoir above the dam was sealed by silt, the uplift pressure on the bottom of the dams on sand and gravel foundations varied from nearly full head at the up-stream end to the tail-water head at the lower end, diminishing more or less closely according to the "line-of-creep" theory. Considering the results of the author's experiments and the corroborative evidence recently published, it appears that his conclusion as to full hydrostatic head in pervious soil is entirely justified.

Charles R. Gow,‡ M. Am. Soc. C. E. (by letter).§—The author has rendered a useful service to the Engineering Profession in this presentation of the information derived from his ingenious and carefully conducted experiments. While it may be true that his conclusions are not entirely new, they are so definitely confirmatory of previous experience along similar lines that there is no longer any valid reason for the assumptions frequently made that ground-water pressures exert substantially less than 100% of the theoretical pressure due to their heads.

A question in this connection which may be entitled to consideration is that of the reduced pressure applied by a structure on the soil due to this hydrostatic uplift. A building with a water-tight basement situated below water level would apparently suffer a reduction in the amount of weight normally applied to the soil by the quantity of water displacement represented and, in many instances, this may be a considerable factor justifying somewhat higher load values than would ordinarily be used.

Likewise it is to be noted that account is frequently taken of the weight of soil removed in the process of excavating a deep basement in determining the net increase in unit loading which the structure imposes on the supporting stratum. The usual practice has been to calculate the weight of material removed according to its weight in free air. If, however, the excavated

* "Upward Pressures Under Dams; Experiments by the U. S. Bureau of Reclamation," by Julian Hinds, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 685.

† *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1349.

‡ Cons. Engr. and Contr., Boston, Mass.

§ Received by the Secretary, June 2, 1928.

material in its natural state is in whole or in part below the ground-water level, it may be assumed that the actual weight removed in the process of excavation is less than this amount due to its immersion in water. Of course, if hydrostatic uplift is assumed to reduce the weight of the structure this factor need not be considered since the two considerations offset one another.

While the title of the paper refers especially to pervious soils, the author discusses briefly some of his experiments with clay instead of sand. As a matter of fact, it is doubtful if there are in Nature any actually impervious soils. Experiments which have been made seem to indicate that even the densest clays will permit some passage of water, although at an extremely low velocity rate. Under these circumstances there would seem to be no good reason for differentiating between the so-called pervious and impervious soils with respect to the ultimate effect of hydrostatic uplift. There are, however, certain aspects of the problem as it relates to the so-called impervious soils which are entitled to mention.

When a basement is constructed within an excavation made in a clay soil, there is danger that surface water may be admitted into the porous back-fill around the outside of the basement walls which may upon occasions supply temporarily a pressure at the level of the under side of the basement floor-slab equal to that of a water-head of the full depth of excavation. Usually, it is assumed that a more or less water-tight connection is made between the floor masonry and the supposedly impervious soil upon which it rests and to a certain extent this is probably true. One can never be definitely sure, however, that water pressure from such a source as that referred to will not find its way into the seam between the masonry and earth and thereby exert its full uplifting influence.

An instance of this sort occurred a few years ago in connection with a mercantile building in Boston, Mass. The basement floor was laid directly on a soil of the type generally recognized as impervious in character. The space surrounding the basement wall was back-filled as is more or less customary with material of porous character consisting of waste plaster, bricks, and the like, with the result that when a heavy rain occurred shortly after the completion of the building, the voids in this loose back-fill were completely filled with water up to the surface of the ground. A few days later the basement floor which had not been designed against uplift was forced upward over the greater part of its area as a consequence of the admission of water operating under this head.

On another occasion the clear-water basin of a large filtration plant in the vicinity of Boston was founded on a stiff blue clay which showed no observable evidence of water percolation through it. After the completion of the structure a heavy downfall of rain occurred leaving a number of deep pools of water in some of the unfilled trenches around the outside walls. No especial thought was given to the matter until a day or so later it was observed that the center portion of the structure was gradually rising and that the area affected was constantly extending in a lateral direction around the point of initial movement. Ultimately, an area 100 ft. or more in diameter was affected with a maximum rise at the center of 3 ft. Holes were then drilled

through the floor of the basin allowing the water pressure to relieve itself by flowing into the interior of the structure. Thereupon the raised portion gradually settled back into its former position without any damage other than a few slight cracks in the concrete.

Upon investigation it was learned that during the preliminary stages of construction a shallow sump had been excavated at this particular location and lateral drains had been made in the clay to lead into this sump any water that might collect. Prior to placing the concrete floor the sump and open drains had been filled with gravel, thus furnishing a blind drain through which water was admitted from the pools pocketed behind the high walls of the basin, thus permitting its uplifting influence to exert itself.

According to eye-witnesses the upward motion of the structure was very slow and as it increased the affected area widened. This would seem to indicate that the water pressure first forced itself under a sufficient area adjacent to the sump to cause a very slight upward motion which, in turn, raised the adjacent floor-slab sufficiently to enlarge the area of pressure application and thereby the movement became progressive.

Whereas it is customary to design floor-slabs of sufficient strength to permit them to transmit the upward pressure to column and wall supports, it is sometimes found to be economical to anchor the slabs into the soil by means of piles or caissons.

About 1921 a 1 000 000 gal. crude-oil reservoir was constructed in South Boston immediately adjacent to the harbor and at a depth considerably below high-water level. As there was no earth covering over the roof of the tank it was absolutely essential to prevent flotation by establishing some form of anchorage. In this instance the underlying soil was clay and a series of concrete caissons were sunk to a considerable depth below sub-grade. They were belled out at their bases while vertical reinforcing rods were embedded for the full length of the caissons and bonded into the floor of the tank. In this manner the necessary resistance against uplift was provided. The same method has frequently been used by the writer to provided intermediate reactions against uplift in floor spans of considerable extent so as to permit of a more economical type of floor-slab design.

The whole subject of hydrostatic uplift has been an extremely troublesome one for many years, and it is a source of gratification to the writer to have so authoritative an expression regarding its influence as that presented by the author.

E. L. Chandler,* M. Am. Soc. C. E. (by letter).†—The large number of experiments conducted, the very considerable range in the magnitude of the experiments, and the painstaking methods followed by the author in assembling the data for this paper, serve to render his conclusions authoritative in treating a subject that heretofore has brought forth much diversity of opinion. Proper allowance for upward pressure acting on a structure founded on pervious

* With Price Bros. Co., Dayton, Ohio.

† Received by the Secretary, June 4, 1928.

material is a feature of design that has been poorly understood, and the information set forth in this paper is most welcome.

The total pressure developed under any structure depends, of course, on two factors: First, the effective area over which the pressure acts; and, second, the intensity of pressure. As for the first, the author's experiments seem to prove that, for a pervious soil, the area subjected to pressure is so nearly 100% of the total area that it would be unwise to assume any lesser value.

The writer has been interested in experiments to determine values for the second factor, by means of apparatus installed in dams during construction. The best data obtained were in connection with the Pinhook Dam at Maquoketa, Iowa. An account of the work done there, with an analysis of the results, has already been given in a discussion by the writer,* and details will not be repeated here. However, it is believed that the subject matter of that discussion may properly be considered in connection with Mr. Parsons' paper.

From a study of those experiments, the writer concludes that, under a dam on a pervious foundation, uplift due to the full hydrostatic head developed should be assumed at the up-stream edge, and that uplift at the down-stream edge should be taken as equal to tail-water if the apron is submerged, or to zero in the absence of submergence. At any intermediate point the uplift should be determined in accordance with the "line of creep" theory, the pressure diminishing uniformly from the up-stream to the down-stream edge in proportion to the total distance between the two edges when measured along the line of contact between the under side of the structure and the underlying material. Such measurements should take into account all cut-off walls and should follow down the up-stream face of the cut-off and then upward along the down-stream side.

It would seem, therefore, that for structures of the type under consideration, the practice of assuming uplift equal to only a fraction of the full up-stream head, or of considering the upward pressure to act on only a part of the base area, is apt to lead to unstable designs. Although it is true that the pressure may not quite reach 100% of the head, and that the effective area for uplift may not be quite 100% of the total base area, it appears unsafe, in the light of information given, to proceed on any lesser assumptions. This would apply only in a modified form for structures on semi-pervious materials, whereas the question of pressures acting on structures on rock must be approached in a different manner.

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1349.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

FLOOD CONTROL ON THE RIVER PO IN ITALY

Discussion*

BY MESSRS. C. MCD. TOWNSEND AND OREN REED.

C. McD. TOWNSEND,† M. AM. SOC. C. E. (by letter).‡—The writer considers this paper the most valuable of the numerous productions on river hydraulics which have been evolved as a result of the flood of 1927 in the Mississippi Valley. While it contains numerous data which would be otherwise unavailable to most readers, its great value arises in that it recognizes that the science of river hydraulics existed in Italy prior to the discovery of America, and that Italian engineers have been engaged for centuries in investigating the laws governing the flow of water in streams.

While they may not have established laboratories satisfactory to the author, they have made practical experiments on the River Po for generations which are of much greater value. They have published and analyzed the results of their experiments, recording not only their successes but their failures. The description of the attempts to restore the river to the Grand Po after the break occurred near Ferrara, should be read carefully by those who would invite a similar catastrophe in the Mississippi River when constructing spillways.

The great value of the observations arises not only from the thoroughness with which they were made, but also from the long period of time which they embrace. German engineers have demonstrated that the construction of levees has not raised the bed of the Rhine below Lake Constance; those of France have arrived at a similar conclusion from an investigation of the flow of the Loire; and the Mississippi River Commission can find no trace of a river fill due to construction of levees on the Mississippi. The geologist, however, ignores these statements and continues to rely on M. Proney's report of the rising of the bed of the Po at Ferrara as recorded by Cuvier, notwithstanding

* This discussion (of the paper by John R. Freeman, Past-President, Am. Soc. C. E., published in April, 1928, *Proceedings*, and presented at the meeting of June 2, 1928), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Col., U. S. A. (*Retired*), Washington, D. C.

‡ Received by the Secretary, May 11, 1928.

the fact that it was denied and ridiculed by Italian engineers at the time it was made. It has also been claimed that the bed of the Yellow River has been raised due to levee construction. It is, therefore, a valuable addition to the science of river hydraulics when so great an authority as Mr. Freeman asserts not only that the raising of the bed of the Po has been "found to be mostly untrue",* but that the reports about the Yellow River "are mostly untrue although not entirely so."†

Another fallacy which has embarrassed river engineers is the assertion of Gustav Wex in a series of papers published from 1873 to 1879, that deforestation and drainage have caused a rise of the bed of the Danube. Schlichting and other German engineers analyzed the discharge of both the Rhine and Danube, and came to the conclusion that the little effect on the bed of these rivers which had occurred, could be ascribed more logically to other causes. The masterly paper by G. Fantoli on "Il Po nelle Effermeridi di un Secolo" conclusively demonstrates that the only injurious results on the regimen of the Po caused by human agencies, have arisen from the diversion of water for irrigation; and that neither levee construction, deforestation, nor drainage has had an appreciable effect on its flow. Similar conclusions have also been derived from an analysis of the flow of the Upper Mississippi.

Frisi and others decided that the troublesome elevations of river bed occur on the tributaries near where they leave the steep slopes of the narrow mountain ravines and enter upon the much more gentle slope across the main valley floor, and that gravel deposits are found in the upper reaches of the river, while in the lower sections the river bed is composed of finer material readily moved by moderate river currents. These deductions explain the cause of the errors committed by geologists, and indicate why they so readily accept, without investigation, any statement that the bed of a river is rising.

It is unquestionable that mountains and hills are eroded during rain storms and that the material thus moved is deposited in the valleys of streams or in the sea at their mouths. The geologist, however, fails to recognize that there exists, near the head-waters of every valley, a zone of deposition where the heavier portions of the eroded detritus are deposited, and that only the portions readily carried in suspension find their way to the sea. Succeeding floods again set in motion the material first deposited, but the grinding which ensues, converts boulders into pebbles, pebbles into gravel, and gravel into sand, and in a relatively short distance gravel-bars are converted into sand-bars capable of being readily acted upon by river currents. This action occurs not only on the tributaries of the Po, but on the Rhine above Lake Constance, and on the hill streams emptying into the Upper Mississippi. It is a rational explanation of the fill noted by the author on the Yellow River,‡ as this fill occurs below a point where the river debouches from a mountain gorge.

As the writer has explained in various pamphlets and statements, most of which have been interred in the *Congressional Record* and the proceedings of

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 959.

† *Loc. cit.*, p. 961.

‡ "Flood Problems in China", by John R. Freeman, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1422.

Congressional committees, the engineer in charge of the improvement of the navigation of rivers is only interested in that section of a stream which has been brought into unstable equilibrium by this grinding process. In this part of a river, the cause of the rise of the bed is its extension into the sea, as discussed by Mr. Freeman.

The writer has inspected both the Po and the larger rivers of France and is somewhat familiar with European literature on river hydraulics prior to the World War. He has had no opportunity to read the report of March 17, 1924,* but he has studied the Atti del Comitato Technico Executivo, Commisione per la Navigazione Interna, decrito 14 Ottobre, 1903, in which, while suggesting the improvement of the Po by the French system of river control, the Commission recognizes the success of the dredging operations on the Mississippi River in maintaining a channel, and proposes further experiments with dredges before definitely adopting the project.

It is to be inferred from the statements made by the author, that the Italian engineers have finally adopted the report of October 14, 1903, after making experiments extending over a period of twenty-four years.

One of the United States Army Engineers recommended a similar project for the improvement of the Upper Mississippi River in 1898.† The improvement of the Missouri River is based on the same principle.‡

The system of river improvement adopted for the Po has resulted from experiments made by M. Fargue on the Loire River in France and by M. Gerardin on the Rhine. It is the system which has received the approval of the Ecole des Ponts et Chaussées and has been fully described in the textbooks of that institution which have been published since 1899.

The assertion of the author that he would improve the project adopted by straightening the lower reaches of the river, indicates a failure to appreciate the principles governing this method of river regulation. He is proposing to substitute the German method of river regulation which the Italians (as he states) have abandoned, and is re-opening a discussion which has agitated river hydraulic engineers in Europe for many years, and, at present, receives little support even in Germany.

Humphreys and Abbot demonstrated§ that river-shortening led to a lowering of the low-water pools above a cut-off, and to an elevating of the river bed below. M. Gerardin was the first to recognize the injurious effects of such flattening of the slopes on the river's regimen, and to show the necessity of maintaining the alternation of pools and bars which Nature has created in a river.

Italian engineers appreciate that navigation would be irreparably ruined if the slopes on the lower river, which are now gentle, were still further reduced, and the slopes on the upper reaches, which are now excessive, increased thereby. They not only maintain the existing bars by giving a curved trace to their works, but they are especially careful to limit the river contraction so that

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 982.

† *Journal*, Western Soc. of Engrs., Vol. XIV, p. 26.

‡ H. R. Doc. No. 1287, 61st Cong., 3d Sess.

§ "Report upon the Physics and Hydraulics of the Mississippi River", 1861.

they will not create a scour on the bars during flood stages. The limitations of the height of their works to 1 m. above low water is not primarily to reduce flood heights, but rather to invite a fill on bars during a rise in the river. When the river falls, the water is directed along predetermined lines so as to create a channel through the bar which will have the depth desired at all stages. It is for that reason that they have adopted the parabolic curve in their trace.

The extensive use of the waters of the Po for irrigation has rendered the problem more difficult than that occurring on the Upper Mississippi or on the Missouri. The curvature which the bank naturally assumes has, therefore, been more generally utilized in improving the rivers of the United States; and less care has been taken to give a prescribed slope to the contracting dikes.

It is noted that sand dikes and training walls used on the Po are approved by the author. Similar sand dikes have been extensively used by the U. S. Army Engineers in recent years on the Upper Mississippi, and have been equally successful on that river, although they are confined to the convex side of bends.

Permeable dikes were also tried experimentally on the Upper Mississippi more than forty years ago, but brush mattress dams were substituted finally because the river in its upper reaches carries little sediment in suspension even during floods. Below the mouth of the Missouri they have been eminently successful, although it was necessary to give them much greater strength than those described on the Po. In recent years, concrete piles have been quite generally substituted for those of wood, between the mouth of the Missouri River and Cairo, Ill.

The general dimensions of the levees on the River Po were known to the Mississippi River Commission before the existing levee section for that river was adopted, and the sections developed by the Germans on the Rhine and by the French on the Loire were also studied. European levees have generally a greater width of crown and less width at the base than the Mississippi River levees, due principally to the fact that European levees are used extensively as roadways.

Formerly, the usual road of the Southern States consisted of undrained soil without ballast which was nearly impassable during rainy weather. It would have been the height of folly to have permitted the tops of levees to be converted into a similar quagmire by the passage of vehicles over them. It was moreover recognized that the ultimate height of levees had not been definitely determined, and that, if macadam roads were constructed at the low grades then existing, they would have to be rebuilt from time to time. The wisdom of the decision is forcibly illustrated by the results of the flood of 1927. Every road which might have been in existence on the tops of levees would now require reconstruction.

The Commission therefore decided to give to the levee such a form as would offer the maximum resistance against water pressure, with the most economical movement of the earth which formed it. This incidentally created a banquette on which a road could be located whenever the progress in road building reached the stage that the road surface was protected from abrasion. The minimum requirements were a width of crown of 8 ft., and a width of

base equal to ten times the probable head of water against the levee. These dimensions were not based on laboratory tests, but on observation of the resistance of the average soil of the Mississippi Valley to filtration. In many localities, not only in the valley of the Mississippi, but also in the valley of the Po, soil is found which is so porous that levees of the dimensions here given are unsafe, but when the muck ditch reveals such a soil, the levee engineers simply enlarge the section by flattening the slopes.

From the writer's inspection of the Po Valley, he gained the impression that the second line of levees were irrigation ditches in which water is carried at considerable heights above the general surface of the ground. He would not, however, question a statement by an authority as competent as Senator Luiggi. He suggests nevertheless that a second line at the same elevation would exceed in cost that of one along the river bank, since the ground of an alluvial valley slopes away from the river. The financial condition of Italy hardly justifies such a luxury.

If the space between the two levee lines is allowed to fill during a rise of the river, it will materially reduce flood heights. Whatever may be the attitude of the farmers of Lombardy, the writer is convinced that the planters along the Mississippi River would never consent to such a proceeding. There are double lines of levees along certain sections of the Rhine, but they conform to the conditions shown on Fig. 10* a front line overflowed at medium stages and a controlling line to regulate extreme floods. The writer has seen a statement in a German publication that this peculiar construction was adopted as a compromise measure to satisfy owners of vineyards who had a theory that the restriction of floods to the front line of levees would have an injurious effect on the grape crop of the adjoining hills.

Science would be greatly benefited if the Treasury of the United States could afford the expenditure and Congress would authorize the establishment of rain gauges throughout this country at as frequent intervals as in the Po Valley. Such data would be extremely valuable for determining the volume of water which causes a flood, when time has been afforded to assemble and digest them; but as a means of flood prediction they would be practically useless. A flood would have passed before a central office could have computed the percentage of flow that would reach the river. The method adopted by the U. S. Weather Bureau for determining flood heights by gauge readings on the tributaries is far more practicable and also more accurate, because it measures the run-off and avoids the necessity of computing the quantity of water evaporated and absorbed by the soil. The flood predictions of the Weather Bureau compare most favorably with those of European nations.

A marked advance in flood prediction was also made by U. S. Army Engineers during the flood of 1927 in the Mississippi River. The determination of the height that the water would attain in the area flooded by crevasses, was of inestimable value to Secretary Hoover in his efforts to rescue flood sufferers, particularly the work of Captain Lewis A. Pick in the valleys of the Teche and Atchafalaya Rivers.

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 978.

The observations of Pontelagoscuro accurately determine the quantity of material carried in suspension by the River Po at different stages.* They however merely supplement the work of the Mississippi River Commission and of U. S. Army Engineers in the investigation of the flow of sediment in streams. The observations on the Mississippi River not only trace the flow of material in suspension from its source in the tributaries to its deposition in the Gulf of Mexico, but also measure the quantity of material moving along the river bed in sand waves. Furthermore, they indicate the effect of slope, curvature, and discharge on the height and permanency of sand-bars, and the relation of caving banks to turbidity.

The author invites attention to the employment of civil instead of military engineers in improving the rivers and harbors of Italy, but he fails to recognize that the Italian civil engineers are taught the fundamental principles of the science of river hydraulics in the various engineering schools which he enumerates, while the Italian Army Engineer does not have such instruction. In the United States, conditions are reversed. The U. S. Army Engineer is taught at least the rudiments of the science at the Army School at Fort Humphreys, while it is very generally ignored in the various engineering colleges of the country. The assistant engineers who have been employed and frequently have been instructed by the Army Officer are the only other body of engineers who have had practical experience with the problems which arise in the improvement of rivers and harbors in the United States.

The author will be of great service to American engineers if he succeeds in his efforts to persuade Italian engineers to translate into English, their knowledge of river hydraulics. With such information available, it is even possible that it will be unnecessary to establish a laboratory in Washington for the purpose of rediscovering principles known in Italy for centuries and now taught in the ordinary textbooks of the engineering schools of Europe.

Oren Reed,† Assoc. M. Am. Soc. C. E. (by letter).‡—With the 1927 flood on the Mississippi River fresh in their memory, American engineers, especially hydraulic engineers, will welcome the author's excellent description of the flood problems of the River Po in Italy. For centuries various methods have been tried in an effort to control the flood flow of the Po, with only partial success. As their knowledge of river behavior has become more complete, the Italian engineers have been able to lessen the damage from floods. Certain practices, which have been proved by long experience to be wise, could be adopted with profit by American engineers.

One point which was not mentioned by the author is the influence of lakes, either natural or artificial, in reducing flood flow. As is shown in Fig. 1,§ the south slope of the Alps receives the largest precipitation of the Po water-shed. This drainage is partly regulated by large lakes—Garda, Iseo, Como, Lugano, and Maggiore. The regulating effect of these lakes would be very great without any artificial control. Lake Maggiore has an

* *Proceedings*, Am. Soc C. E., April, 1928, Papers and Discussions, p. 963.

† Asst. Designing Engr., San Joaquin Light & Power Corporation, Fresno, Calif.

‡ Received by the Secretary, June 9, 1928.

§ *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 960.

area of about 83 sq. miles. A record of its maximum and minimum water levels shows that the yearly variation is from 6 to 16 ft.* This large annual variation would indicate that in a wet year the lake had temporarily retained a large volume of water which would materially affect the flood discharge of the Lower Po.

The effect of natural retention in lakes was clearly illustrated in September, 1927, by Lake Constance, which is located on the German-Swiss border. The head-waters of the Rhine River in Switzerland were swollen by a severe two-day storm in the last days of September, 1927. The quantity of water carried by the Rhine above Lake Constance on September 25 and 26, 1927, was the greatest yet observed. The flow into Lake Constance from the Rhine on September 25 reached a maximum of 81 300 sec-ft. The total inflow into the lake was 148 500 sec-ft. The maximum outflow occurred on September 29, and was only 29 700 sec-ft.†

In Norway, investigations have recently been completed to establish a comprehensive plan for flood regulation on the Skien River. This study was prompted by a flood of high intensity in June, 1927, which caused great property loss. The communities affected by the high water, appointed committees for the purpose of perfecting a plan for preventing a similar catastrophe in the future. The technical study was under the direction of Christian Raestad.

The Skien is the third largest water-shed in Norway and has a drainage area of 4 060 sq. miles at the City of Skien, which is about six miles from the North Sea. Nearly all the area is mountainous and includes numerous lakes and fjords. The head-waters flow from a plateau region, a large part of which is marsh land.

The Skien District is one of the most important in Norway. It has long been noted as a timber-producing district. "Norsk Hydro" (the Norwegian Electric Company) is located in the central part of the water-shed about 65 miles from the ocean. This Company, the leading industrial concern in Norway, uses hydro-electric power for the fixation of atmospheric nitrogen. The total plant capacity controlled by Norsk Hydro amounts to about 260 000 kw.

Dams have been constructed at the outlets of a number of large lakes in the Skien water-shed for the purpose of regulating the flow for power and for the floating of timber. Most of these regulated lakes are in the upper part of the water-shed. The most important ones are: Mos Lake, 597 000 acre-ft.; Maar Lake, 183 000 acre-ft.; and Tinn Lake, 159 000 acre-ft.

Authentic records of floods on the Skien River begin about 1645. Serious floods have occurred at frequent intervals, but none has caused as much property damage as the flood of 1927. The banks of the river had been encroached upon in recent years by industries and private interests. After the regulation of Mos and Maar Lakes the low-lying areas were built upon, for it was expected that flood levels would be reduced. The 1927 flood was caused by heavy precipitation, augmented by melting snow, on the lower part

* *Schweizerische Wasserwirtschaft*, February, 1928, p. 21.

† *Loc. cit.*, p. 17.

of the water-shed. Regulation of the upper lakes decreased the possible flood, because no water was released from Mos Lake during the flood period.

Floods on the Skien have usually occurred in the late spring and are caused, as in 1927, by heavy precipitation and melting snow. There have been, however, many autumn floods. The highest recorded flood at the City of Skien was in June, 1927, and amounted to about 106 000 sec-ft., or 26.2 sec-ft. per sq. mile of total area.

The recommendations of the Committee appointed in 1927 were: (*a*) to increase the capacity of the stream at critical points; and (*b*) to decrease the flood flow by further regulation of natural lakes.

At the City of Skien and at Skot Fall, crib dams serve for diversion to low-head power plants. During flood periods these dams materially raise the water level. It is proposed to remove these crib dams and construct new structures, regulating the flow by means of roller or sector gates. During floods, the gates would be opened and would present no obstruction to the flow. At certain points considerable dredging will be necessary to obtain sufficient channel capacity for the maximum possible flood.

The flow in the upper part of the drainage basin is regulated at present in Mos, Maar, and Tinn Lakes. Additional regulation is planned for the middle section of the water-shed at Lake Totak and at Sundsbarn. It is thought that the maximum probable flood can be economically decreased about 21 000 sec-ft. by the proper regulation of these lakes. This would be about 20% of the 1927 flood.

By regulation more of the flow can be made useful for industries and power plants. This will be a valuable feature, although the chief purpose of the work will be a reduction of flood flow. It is thought that the work can be financed by assessments on the communities and industries benefited without any aid from the State. For reasons of economy, it will be necessary to do the work in two or more steps.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

QUALIFICATIONS OF CONTRACTORS ON PUBLIC WORKS

Discussion*

By Messrs. Ralph F. Proctor, Walter S. Anderson, Morris Knowles, H. C. Boyden, and T. Chalkley Hatton.

Ralph F. Proctor,† M. Am. Soc. C. E.—It is the duty of the contract bond underwriter for a surety company, to select those contractors whose qualifications measure up to his company's standards on any particular piece of work.

There are certain general qualifications which every successful underwriter keeps in mind in the consideration of each and every case presented to him. In brief, these are the moral hazard; experience and competency as shown by past experience; adequacy of organization and plant for the particular work under consideration; amount and kind of uncompleted work on hand at the time application for the new bond is made; tendency of the labor and material market to go up or down; the length of time that the job will take; the amount of the contract to be sub-let and the qualifications of subcontractors; and, finally, the sufficiency of the contractor's financial resources for his work under construction and the new work to be undertaken.

The satisfaction of these general qualifications on any particular case should entitle the contractor to the bond and the surety should feel reasonably certain that no trouble on the case would ensue. In actual practice, however, trouble and loss are experienced by the surety companies even if an underwriter has applied these general qualifications from information available and has found them to be satisfactory. This may be due to a number of causes, such as incorrect information presented, labor troubles, change upward of labor and material prices after the job is let, long periods of unfa-

* This discussion (of the paper by Frank T. Sheets, M. Am. Soc. C. E., presented at the meeting of the Highway Division, Denver, Colo., July 14, 1927, and published in April, 1928, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Fourth Vice-Pres. and Head of Fidelity and Surety Depts., Maryland Casualty Co., Baltimore, Md.

vorable weather and unexpected floods, necessary changes in organization in the midst of the work, and unexpected physical difficulties with the work itself. All these hazards and many others are well known to engineers, contractors, and sureties alike. Construction work is fundamentally a hazardous business because all the risk of unforeseen and unknown conditions belonging to the owner, is assumed by the contractor when he bids to accomplish the undertaking at an upset price or at a unit price. If all the contractors were gifted with an exact foresight, or if the sureties were likewise gifted and could pick only those contractors who were qualified and who would succeed, there would be no defaulted contracts and no surety company would be in the business of writing contract bonds.

Bond underwriters make mistakes in judgment, the same as business men in any other walk of life, including the contractors. It would be idle to deny that surety companies have not issued bonds for contractors who afterward proved unworthy of suretyship. The speaker's experience, however, indicates that most of the bonds written, and on which trouble and loss developed, were for contractors who (on the facts presented) were entitled to surety coverage at the time the bond was given. A general impression prevails that surety companies execute for unqualified contractors a far greater number of bonds than is actually the case. During 1926 the Maryland Casualty Company executed 19 638 contract bonds of all kinds, while the number of bonds on which trouble was reported that year was only 291, or approximately 1½ per cent. Many of these trouble cases will ultimately result in no loss. Certainly no criticism can attach when the company's underwriting organization shows such results.

It is not the multitude of small contract bonds that makes up the major part of the surety companies' loss ratio. It is the failure on large contracts covered by large bonds and undertaken by contractors generally recognized as thoroughly responsible. In order to make up for a loss of $100 000 on a contract bond at the present rate of 1½% on the contract price, a surety company must execute bonds covering work costing $11 500 000, and this will only make it break even, without a cent of profit. In the building of the New York State Barge Canal, the losses to the sureties on work undertaken by supposedly responsible contractors ran into hundreds of thousands of dollars. The Shoshone Dam alone cost the contractors' surety $320 000 which (to make up without profit) would necessitate the execution of bonds covering construction work of approximately $35 000 000. It is not generally known that, to complete the Bear Mountain Bridge, the surety companies put up $1 000 000. The speaker has never seen any article in engineering and contracting publications giving the sureties credit for the part they played in completing this wonderful piece of work. Yet no one can deny that they played an important part in a critical situation without a moment's delay or argument.

The Shandaken Tunnel contract is another striking example of a responsible contractor, whose difficulties made it necessary for the sureties to step in and assume the financing of the work to its successful completion. So skillfully and promptly was this done that "with new capital and efficient

management the new organization [the Shandaken Tunnel Corporation] succeeded in overcoming the time handicap and completed the underground work four months ahead of schedule".*

The Shandaken Tunnel Corporation was quickly created by the surety companies at the time default occurred, in order that they might fulfill their part of the contract set forth in their bond. This Corporation was, in fact, the surety companies themselves, having been formed by them as a vehicle to enable them to carry out their obligations because their charters do not permit them to enter the contracting business in their own name. The sureties then selected the Ulen Contracting Corporation to complete the contract.

It is failures such as these, and many more too numerous to mention, which are in the back of the mind of the underwriter as he sits at his desk passing every working day on submissions to his company. It is not an easy job, as the speaker knows from experience. Who could have foreseen the failure of The William Cramp & Sons' Ship & Engine Building Company; and yet that Company found itself hopelessly insolvent, with three large shipbuilding contracts on its hands, running into millions,—all of them covered by surety bonds. The burden of finding ways and means of completing those contracts fell on four surety companies. Working rapidly and efficiently with all the interests involved—bankers, stockholders, owners for whom the ships were being built (including the U. S. Government), material men, and everybody down the line—a plan was devised for completing the unfinished contracts at the original cost to the owners and without putting the Cramp Company in the hands of receivers, although the plan necessitates its eventual liquidation. This was accomplished by complete co-operation throughout by all interests involved; by the putting up of large sums of money and the establishment of material credits by the sureties and the stockholders; and by extension of time by the banks and the carrying on by the sub-contractors with the assurance of ultimate payment. The sub-contractors were paid almost immediately, because the new cash and credits made it possible to take care of them fully without further delay.

Before the surety companies noticed this trouble, the credit of the Cramp Company was considered to be of the best. The Company was in the preferred class of responsible contractors of this country. It had extended credit with banks, material men, sub-contractors, and sureties. Only three months prior to this trouble it secured, easily, a bond on a large piece of work. Its officials themselves did not know that the Company was in trouble, as it must have been, when this bond was secured.

Thus, it must be evident that the underwriter, no matter how skilled he may be in putting the proper value on a contractor's qualifications, may find himself and his company confronted with a serious loss before the bond obligations are terminated. The contractor may have all the qualifications necessary at the start of an undertaking and still default in his performance, as has been proved by experience time and time again. That is the risk the surety must take.

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* *Proceedings,* Am. Soc. C. E., May, 1927, Papers and Discussions, p. 695.

In almost every case the underwriter is forced to take the lowest judgment of the cost of the work if he is to execute any bonds at all. The lowest bidder usually is awarded the work, and the choice, therefore, is usually against the underwriter. A very recent example of this fact is the results of the bidding on the Fulton Street Tunnel, in New York, N. Y. The lowest bidder was \$2 000 000 lower than the next highest bidder, and \$7 733 000 lower than the highest. The bids ranged from \$22 467 000 to \$32 200 000. The work is estimated to take 5 years to complete. A bond of \$3 000 000 is required, to write which, it will be necessary for almost every surety company in the business to carry a portion of the risk. To decide as to the qualifications of the low bidder on this work to carry it successfully to completion of the surety companies have the advantage of the judgment of most of the oldest and most successful underwriters. The contractor has only his judgment and that of his organization as to the sufficiency of his bid. The sureties also have the judgment of seven other contractors as to what the cost should be. The important question is whose judgment is correct among the eight bidders. Is the low bidder qualified to undertake the work on the basis of his bid, assuming that he is experienced in this line of work and that he has a fair amount of liquid assets for a job of this size, if everything turns out as expected? The selection of contractors entitled to a bond, based on their qualifications, is not an easy one. In fact, it is a serious one, which requires a thorough investigation of facts, and a careful weighing of those facts after they have been obtained.

It is obvious that the right price is one of the fundamental qualifications for the successful completion of a piece of work. The underwriter, in passing on a low bidder's qualifications, derives a considerable degree of comfort when the bidding is close and the contractors bidding are known, from years of experience, to be qualified to undertake the particular work in question. A comparison of the bids is an added check and an aid to the underwriter. Where bid bonds, or bid letters are required, the underwriter is denied this check, and the sureties strongly advise against the use of bid bonds and bid letters. The ability to obtain a certified check proportionate to the size of the contract is in itself one evidence of responsibility and credit enjoyed by the contractor.

It is easy to pass on cases which are obviously so good that there is no question about them, or which fall so low as to qualifications that they can immediately be declined. Unfortunately for the underwriter, however, most of the applications for bonds come in the class which are neither so good that this fact is immediately apparent, or so poor that with justice they can at once be declined. It is just as grave an error on the part of an underwriter to decline a case that is entitled to a bond, as it is to accept a bad risk.

During the last three or four years, there has been considerable published criticism of the methods of underwriting used by the surety companies. Perhaps it would be more correct to state that the criticism is directed at the lack of proper underwriting of contractors' qualifications in issuing bonds. The speaker does not say that the companies deserve no criticism, but much of it has been unfair and caused by lack of knowledge of the real problems

confronting the underwriters. It is certainly not true that the surety companies are solely responsible for defaulted contracts because some company will issue a bond for any contractor whether or not he is qualified. The companies are not looking for losses, but for profits, and no underwriter would execute a bond if he thought a loss might occur. The following factors have been listed in the technical press in answer to the question, "What is the matter with contracting?":*

1.—Ignorance of costs.
2.—Poor estimating.
3.—No distinction of class.
4.—Inadequate engineering.
5.—Hazards in plans and specifications.
6.—Poor judgment of wage and price markets.
7.—Disposition to gamble.
8.—Need of co-operation and organization.

If these are the conditions in the contracting field, what about the poor underwriter on whom falls the duty of separating the wheat from the chaff. Is it to be wondered that his judgment cannot always be 100% perfect? However, it will be found that his judgment is remarkably accurate and his mistakes astoundingly few, considering the total number of cases passing over his desk in a year.

What surety campanies need more than criticism to reduce their mistakes in judgment to the irreducible minimum, is a correct understanding of underwriting methods and problems by the engineers, architects, and contractors, and assistance in collecting the true facts on which to base an underwriting judgment.

Considerable progress has already been made along these lines in the last few years by joint meetings between committees of the Associated General Contractors of America, the Association of Highway Commissioners, the American Institute of Architects, the Surety Association of America, and the Society. As a result of these meetings, there has been developed a Uniform Application and a Uniform Financial Statement, the use of which (by all interested parties to a contract) has already been fruitful in aiding in the elimination of the unqualified contractor.

The qualifications of the responsible contractor, as promulgated by The Associated General Contractors of America are as follows:

"The Responsible Contractor,

"Whether an individual, firm or corporation, must possess as a minimum of requirement, these essential qualifications as follows:

"Skill

"He must possess the necessary technical knowledge and practical experience, as applied to his particular form or group of undertakings, to enable him to carry them to completion in a workmanlike and economical manner.

"Integrity

"He must consistently and persistently comply with the spirit as well as the letter of his contracts. He must have business experience and handle every transaction with fairness and honor.

* *Engineering News-Record*, v. 95, No. 12, p. 456, September 17, 1925.

"RESPONSIBILITY

"He must possess cash or credit to meet all his commitments; also the equipment and organization for the satisfactory performance and completion of his undertakings.

"In special cases, problems may appear which will demand that consideration be given to other qualifications, but in general the above three requirements apply to all construction contracts and without them no contractor can properly be regarded as wholly responsible."

WALTER S. ANDERSON,* ASSOC. M. AM SOC. C. E.—According to the author, the fundamental prerequisites of a contractor are organization and finance. These are identical with those required in all lines of commercial endeavor. Contracting is no more and no less than a business venture. The bald essentials are money and skill. In general, proper admixtures of these two will bring success to business men and hence to contractors, large and small.

Contracting is a highly specialized business, but the prime rules of general business apply as forcibly to it. Contractors cannot divorce themselves from other business men; they must not shun the lessons taught by others; and they cannot ignore the precepts wrought from other lines of business.

Money is essential and the amount needed is determined by the skill of the management. Management, therefore, is superior to money. Management is the organization provided by business men. The management determines the volume of business to be done. The relation between volume of business done and capital available is one of the important factors that spells success or failure in business, as this relation determines the number of times each year capital is turned over. Failure follows if too great a turnover is attempted.

Volume of business depends on the law of supply and demand. This volume will, if the concern is ably managed, consist only of contracts that are profitable. An able management never takes a contract that will result in a monetary loss, although a loss may occur due to unusual mishaps, acts of God, or facts that could not possibly be determined or known at the time the contract was made. However, the unknown facts of large consequence, in general, are very few in number and if ever the unknown facts become so vital as to preclude failure, the risk should then be taken by the owner and not by the contractor. Otherwise, the contract becomes a gamble and is no longer a business venture.

Capital is the sum total of the actual cash and bank credit available. This total is increased with success and wiped out entirely by failure. Success is brought on by an able management that has the necessary knowledge and experience. Failure is brought on by a management that lacks this knowledge and experience, but knowledge or experience alone cannot make a business continue to prosper even if the capital available is sufficient. Lasting success can only be achieved through the personal qualities of character and stability. Character entails honesty, integrity, consideration for others, and righteousness. Stability is that quality which gives men power to resist temptations to over-extend their operations, and to go beyond their finances.

* Vice-Pres. and Gen. Mgr., J. J. Dunnegan Const. Co., Chicago, Ill.

Stability preserves and maintains an organization for the one and only set purpose of making a legitimate profit and being of lasting service to the public.

All failures in the field of contracting have been due to a lack of one or more of the four qualities noted, but a large percentage of failures is due to lack of the second quality, that of stability. Stability is urgently needed when competition is keen, when the demand for work is greater than the supply. At such times the rank and file of contractors show a lack of stability or the ability to resist the temptation to work at an extremely low margin of profit, thereby inviting failure. Stability is vital at these times in resisting the temptation to sneer at former cost figures and to entertain the hope and belief that such costs can be lowered on that new job. To omit entirely or to reduce appreciably the former charges for machinery depreciation, overhead, and contingencies so as to make an estimate show a small profit, is the prevalent practice. Such practice proves that stability is absent, that the contractor does not want to face the facts; whereas stability would demand that these remain the same, and, consequently, the estimate would show no profit, leaving the contractor then to decide whether or not to take work at no profit.

Contractors who lack the qualities cited are on the road to failure at some future time, and, of these, stability is the one main quality that they lack. They fool themselves first, and then fool their bankers; and, finally, when failure arrives, they console themselves by telling the world exaggerated tales about the hazards of contracting, building a romance around their own failure, when, as a matter of fact, if they had been honest with themselves and with the public at large, they would have attributed their failure to not adhering to one or more of the essential qualifications of all successful contractors, namely, character, stability, knowledge, and capital.

The writer has presumed that all engineers are honest and fair. Without these attributes on the part of the engineer, an able contractor cannot profit on the job in question. To an able contractor, an engineer is a barometer and the engineer's degree of fairness, tempered with experience, foretells the percentage of profit or no profit. An engineer "can break or make a contractor". In him a great responsibility rests. Hence there must be the most complete co-operation between the contractor and the engineer. An able engineer and an able contractor will always execute contracts to the lasting satisfaction of themselves and the public.

Morris Knowles,* M. Am. Soc. C. E.—Contractors and engineers alike are much interested in the subject of "Standard Questionnaires", as presented by Mr. Sheets,† particularly relating to experience, capacity, equipment, and those factors which help to determine whether a contractor is an able man, or whether he has too many things on hand to undertake successfully a new job. The financial questionnaire is also a valuable adjunct to that dealing with construction matters.

* Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

† *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1023.

What is to be done, however, with the information from the questionnaires after it is obtained? Of course, publicity is very helpful; but, after all, occasions have been found where it is so hard, where the decision must be made—either by a person elected by the people, or by a group elected by the people, or by some one appointed and on whom influence will bear—not to let friendship and sometimes political favor and other things enter into the decisions in regard to public works. There are a few occasions where such difficulties arise with private undertakings.

Attention has been called to the fact that public officials are besieged with appeals to the effect that "the contractor is hard up", and that "if he can be helped out, he will be put upon his feet". Every engineer has heard all these remarks; so it is not easy to make a decision, even after full information is obtained.

Recently, after a certain city in Pennsylvania had received bids, one man, who was second lowest bidder on one contract and who did not bid on a second contract, offered privately to the City Council to do both contracts for $500 less than the sum of the two lowest bids on the two contracts. He happened to be a local man, and members of the City Council felt that a local man should be favored if, in the opinion of the City Solicitor, it could be done legally. Strange as it may seem, the Attorney decided as Council expected him to do.

This happened to be a case where the engineer was not obliged to participate in the construction, because he was asked to prepare plans and specifications only, with the idea that, later, the City would make some arrangement as to supervision. He decided, therefore, to withdraw from the situation. This caused publicity that attracted the attention of the Associated Pennsylvania Constructors and, through a taxpayer, a suit was brought. The decision of the Court was that the contracts, thus awarded, were illegal.

It is not always possible to withdraw, as some may have found. If an engineer agrees to undertake some work, and that work includes the item of inspection, he may not properly and legally withdraw, and he is obliged to go through with it. He may desire very much to call attention to the fact that error has been committed, that the work is not being done properly, and that a contractor has been chosen who is not able and not financially situated to finish the work. However, no matter what the situation may be, he must go through with the program.

Most engineers like to supervise the work they have designed. It is the proper thing to do, and it ought to be done, as they have the most knowledge of the plans and specifications and have a great deal of interest in seeing that the work is done properly. Nevertheless, there may be times when, for their own self-respect as well as the good of public work in general, they should, where possible to do so legally, withdraw and call public attention to the wrong in the award.

Just for the purpose of presenting something for discussion, a clause is offered, which can be inserted in the contract with the client, the municipality, county, or whomsoever it may be, which may, in some degree, afford a way out. Many will criticize it as being too bold, as putting undue responsibility on the

engineer, and possibly, if he is not honest, affording opportunity for collusion. Nevertheless, engineers should state, first, that they do agree in a concentration of responsibility; and, second, that they are perfectly willing to see it concentrated in the engineer. Then why not face the situation boldly? It is customary, in making arrangements for an engineer, where he is to be paid certain sums of money for the execution of the work, that he receive his money periodically. Furthermore, that after preparing plans and specifications, he shall place men on the job and look after the inspection of various parts.

In order to afford the chance for withdrawal and to give public notice of a wrong, it is suggested that a clause be placed in such contract with the client which reads as follows:

"The engineer shall make an investigation of the history, reputation, organization, and capacity for satisfactory and faithful performance of work of similar character, of all the bidders for the whole or any part of the work and shall certify to the city (or other client as the case may be), such of them as may be found qualified and experienced successfully to perform this work. This certification or report shall contain a statement of the reasons why the bidder is approved or disapproved by the engineer. The city [this is important] shall select the successful bidder or bidders from those thus certified as qualified by the engineer".

It is natural that the determination of qualifications shall be based on the questionnaire that has been mentioned by the author. Answers to the questionnaires will be received with bids and a report made promptly thereafter. What is the result? Of course, nobody can take the place of the city council, or the director of public works, or the men legally chosen to let a contract. That goes without saying. It is not possible to transfer the responsibility of determination of award to the engineer. He can have no such authority; he has no such power. Nevertheless, in case of such clause in his contract, he has a responsibility to report. He has a duty to study these questionnaires and make the investigation, and then to make a report. It takes courage, of course, to do so; but then what happens? The client must choose from the contractors, so carefully studied and certified as qualified, a successful bidder. Suppose that they do not. The engineer immediately can withdraw from the work, and pitiless publicity is frequently the only way out and the only successful corrective of abuses.

This is offered as a suggestion, and with the hope that it will be discussed and tried.

H. C. Boyden,* M. Am. Soc. C. E.—The relations between engineers and contractors are not always based on a thorough understanding of the other fellow's problems and anything that will bring about a more general understanding on both sides will be of distinct benefit to the construction industry. Just where the fault lies and what the remedies are is, of course, a matter subject to a variety of opinions.

Taking a view of the matter, based on many years of construction experience, it appears that there is much to be said on both sides of the question. It has often been stated "that the engineer can either make or break the

* Lecturer, Celite Products Co., Los Angeles, Calif.

contractor" and a careful analysis of most contracts and specifications bears out this statement. Due to this fact there is a feeling of uncertainty in the mind of the contractor during the progress of the work and this is often reflected in the bidding and in the manner in which the work is done. This feeling of uncertainty is often of sufficient importance to cause contractors to refuse to bid when certain engineers are to be in charge of the work, or else the bids are raised to cover the risk.

On the other side, the causes that have led to the writing of contracts and specifications in such a manner as to justify the remark previously quoted, can only be laid to the fact that advantage has been taken so often by irresponsible contractors of more loosely drawn contracts. The remedy for a condition that has taken many years to develop cannot be put into effect in a day, but it will require much careful study and mutual effort on the part of all those who are involved.

As an important step in bringing about a better condition of affairs, the speaker suggests that every engineer analyze carefully all his contracts and specifications, with one basic thought in view. No clauses should be included that are not to be rigidly enforced and until such practice becomes universal a statement to that effect should be emphasized in every tender for bids.

Attention is called to the fact that, where standard general specifications are used, as in State highway contracts, they necessarily cover all types of construction and very often certain clauses are not applicable to the particular contract at hand. Unless care is taken to eliminate such clauses, there is always doubt in the minds of the bidders as to whether or not they are to be enforced, and doubts lead either to high bids, or to gambling on the engineer's interpretation of the contract.

In one case of record, such a clause, had it been enforced, would have broken the contractor without serving any useful purpose whatsoever. In spite of this fact, it was only by strenuous efforts that the clause was revoked and the contractor relieved of the necessity of doing work that was not intended to be done on this particular contract.

Let the engineer be the first to start the remedies to correct the present unsatisfactory condition, and the contractor will soon fall in line and co-operate with him.

T. Chalkley Hatton,* M. Am. Soc. C. E.—The speaker has frequently heard the expression, made before public bodies considering contractors' claims, that "an engineer can make or break a contractor". This implies dishonesty and injustice to the members of a profession which has long been considered as particularly trustworthy.

It must be admitted that not all the members of the Engineering Profession are both honest and just, but the percentage of deficiency is very low. The underlying trouble in public contracts is the system (provided by statute usually) that contracts must be let to the lowest responsible bidder and that, in most cases, the responsibility is measured by the contractor's ability to furnish surety bonds.

* Cons. Engr., Milwaukee, Wis.

In the forty years during which the speaker has had charge of public works there has been but one instance when the lowest bidder could not furnish satisfactory bonds, and yet there have been many in which the contractor failed to complete his contract, and some in which the sureties likewise failed and the burden was carried by the public.

In most of these cases the lowest bid was much less than the engineer's estimate and much below the average bids received; and still the sureties were willing to take the risk. The evidence in the usual case seems to have shown conclusively that the surety company assuming the risks required that the contractor, or his friends, give ample supplementary security; or, that the contract need not be carried out in substantial compliance with the specifications. Thus, the surety company's risk was zero.

In a public contract there are only two parties, the public and the contractor; the surety is substantially a part of the latter. There must of necessity be some agency to determine that one party gets what he is paying for and that the other gives what he is being paid for. The usual public contract stipulates that the engineer representing the public be that agency. To control his actions, plans and specifications are prepared in advance of the bidding and these form a part of the contract. If they are not sufficient to enable either party, or the engineer, to determine what is to be expected of each, no contractor should submit a bid; and if he does, no responsible sureties should become his surety. If they do, they cannot claim that the engineer broke them because, under the common law, he can only see that the terms of the contract are complied with as he interprets them.

It may be true that he errs in judgment, but he usually exercises his best understanding, not with a view of breaking the contract or to enrich the public, but to do justice to all parties; if the contractor believes he has not been treated justly he has "his day in Court."

The expression of the engineer being able, "to make or break a contractor" arises from the old idea that a contractor may, and expects to, skimp his work and bids accordingly. When he finds that the engineer intends to interpret the specifications justly, he raises this "howl" which engineers so often have heard in the past.

Thanks to the Associated General Contractors of America, a different type of contractor is being produced and a high standard of ethics is being adopted. This will result in a better grade of all public works with less trouble to the engineer and more profit to the contractor, and probably to the surety companies. However, the speaker believes that the latter should mend their ways to the extent of refusing to give bidding bonds, thus being quite free to accept or reject offers to become surety for any contractor until they have had ample time to study the bid submitted by him.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

HYDRAULIC STUDIES AND OPERATING RESULTS ON THE MIAMI FLOOD CONTROL SYSTEM

Discussion*

By Messrs. C. S. Bennett, Morris Knowles, and Walter M. Smith.

C. S. Bennett,† M. Am. Soc. C. E. (by letter).‡—Some details of the manner of obtaining and recording data for use in studies of the operation of the flood control system of the Miami Conservancy District may be of interest. In order to systematize the work it became necessary to adopt certain standard methods for making observations and to develop forms on which to record the data gathered.

On the map of the drainage area, Fig. 12,§ are shown the stations at which records of rainfall or run-off are obtained. Of the stations shown, seventeen report directly to the District Office. The forms shown in Figs. 16 and 17 are used in connection with recording rainfall and river-stage readings. They are printed on 3 by 5-in., U. S. Post Cards. The cards are mailed to the District Office by the observers at the end of each week. Cards of this size may be filed in standard card files and are thus readily available for reference. All observers at rainfall stations are instructed to telephone the Office whenever a rainfall of 1 in. or more occurs in 24 hours.

Studies are made of every major storm that occurs. To facilitate the collection and use of data for these studies forms such as Figs. 18 and 19 were devised. Fig. 18 is used by the observers at the dams. From the time that the outlet conduits are half full until the stage falls again to that elevation, readings are made every 2 hours at the gauges above and below the dam. A series of staff gauges have been placed on or near the up-stream slope of each dam at convenient locations, the range of these gauges extending from the

* This discussion (of the paper by C. H. Eiffert, M. Am. Soc. C. E., published in May, 1928, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Engr., The Miami Conservancy Dist., Dayton, Ohio.

‡ Received by the Secretary, June 6, 1928.

§ *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1386.

elevation of the mid-point in the conduit to spillway crest elevation. Below the dams are automatic recording gauges, near which staff gauges are located in the stream. The latter are read by the observers during times of high water, such readings serving as a check on the automatic recorder. From these it is possible to construct inflow and outflow curves for the storms to be studied.

THE MIAMI CONSERVANCY DISTRICT

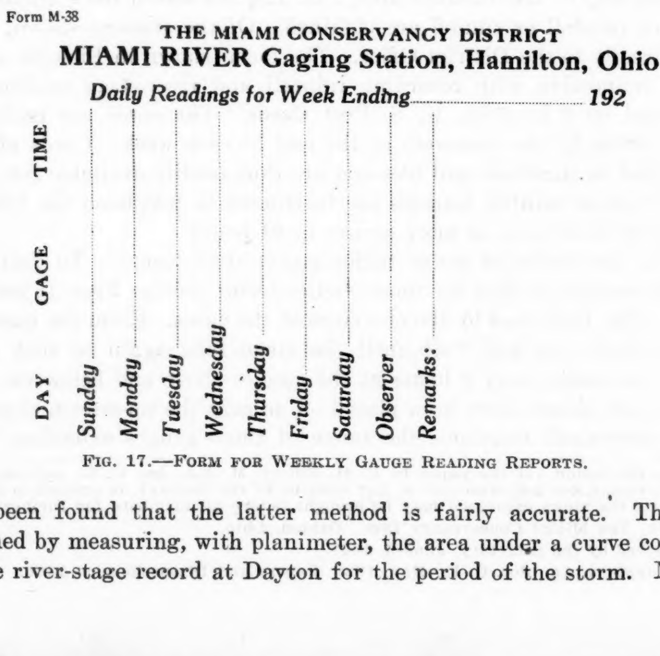
RAINFALL AT ________ FOR WEEK ENDING ________ 19____

DATE	TIME OF BEGINNING	TIME OF ENDING	AMOUNT	REMARKS

OBSERVER ____________________

FIG. 16.—FORM FOR WEEKLY PRECIPITATION REPORTS.

On the form, Fig. 19, are recorded the essential data for each storm. In connection with this a rainfall map is prepared. The observed rainfall at each station is noted on the map and rainfall contours or isohyetals are drawn in. The average rainfall and run-off are computed usually only for the Dayton Station. Average rainfall may be obtained by planimeter from the contour map or by using the mean of the recorded values above Dayton.


Form M-38

THE MIAMI CONSERVANCY DISTRICT

MIAMI RIVER Gaging Station, Hamilton, Ohio

Daily Readings for Week Ending 192

DAY | GAGE | TIME

Sunday
Monday
Tuesday
Wednesday
Thursday
Friday
Saturday
Observer
Remarks:

FIG. 17.—FORM FOR WEEKLY GAUGE READING REPORTS.

It has been found that the latter method is fairly accurate. The run-off is obtained by measuring, with planimeter, the area under a curve constructed from the river-stage record at Dayton for the period of the storm. Maximum

GAGE RECORD FOR____________________ DAM.

Date______			Date______			Date______		
TIME	UPPER GAGE	LOWER GAGE	TIME	UPPER GAGE	LOWER GAGE	TIME	UPPER GAGE	LOWER GAGE
Date______			Date______			Date______		
TIME	UPPER GAGE	LOWER GAGE	TIME	UPPER GAGE	LOWER GAGE	TIME	UPPER GAGE	LOWER GAGE

At what time were conduits just full________________

What was highest point reached on upper gage________________

When________________

What was total rainfall for this storm________________

When did it start________________ When did it stop________________

FIG. 18.—FORM FOR GAUGE READING RECORDS.

Storm of______

	ALL STATIONS					RIVER STATIONS ONLY				DAMS ONLY									
LOCATION	Drainage Area Sq. Miles	Rainfall Average Above Station	Runoff in Inches	Max. Rainfall 24 Hrs.	Max. Runoff 24 Hrs.	Max. Gage Height	Time	Max. Disch. Second Feet	Max. Disch. Sec. Feet Per Sq. Mi.	Maximum Inflow Sec. Ft.	Time	Max. Out Flow Sec. Ft.	Time	Max. Elev. W.S. Above Dam	Time	Max. Elev. W.S. Below Dam	Time	Max. Veloc. in Conduits	REMARKS
Germantown	272																		
Englewood	651																		
Lockington	255																		
Taylorsville	1133																		
Huffman	671																		
Sidney	555																		
Piqua	842																		
Troy	908																		
Tipp. City	1016																		
Springfield	488																		
Dayton	2525																		
Miamisburg	2720																		
Franklin	2785																		
Middletown	3162																		
Hamilton	3672																		

FIG. 19.—FORM FOR RECORDING STORM DATA.

run-off in 24 hours may be obtained also from this curve. Maximum discharges at the various river stations are obtained from the rating curves for those stations. Maximum inflow and outflow at the dams are obtained from curves prepared from data contained on the observers' record (Fig. 18) and from the rating curves at stations just below the dam.

Recently, a number of silt-catching boxes have been installed in two of the basins near the dams.* Soon after the subsidence of stored water after each major storm, measurements are made of the silt content within the boxes and on the projecting bases. The data thus obtained are recorded on the form similar to Fig. 20 and these are filed for future use. A number of cross-sections have been taken across the valleys near the silt boxes. At intervals of a year or more, these sections will be repeated and platted to a fairly large scale so that the quantity of silt remaining as a permanent deposit may be measured.

Data on Silt Deposits

Location_______________ Date of Storm______

Max. El. of Backwater______ Time of Max. Stage______

Condition of Ground

Very Dry – Dry – Wet – Very Wet – Frozen

Box No.	Elev.	Duration of Overflow	Depth in Box	Depth on Board

FIG. 20.—FORM FOR RECORDING DATA ON SILT DEPOSITS.

Discharge measurement notes, rating tables, records of daily discharge, and other data secured in connection with stream-flow records are arranged on standard forms of the U. S. Geological Survey, because such information is used jointly by the Geological Survey and the District.

MORRIS KNOWLES,† M. AM. SOC. C. E. (by letter).‡—This paper is an interesting contribution to the already voluminous literature on flood protection works. Its value, however, lies in the fact that it presents actual data on results of works already constructed. There has been a paucity of these

* *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1346.

† Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

‡ Received by the Secretary, June 12, 1928.

data and they are the kind of which the profession should have more, as a guide to the design of future plans of controlling devastating floods.

As the author states,* the period of six years since the completion of the Miami System is not long enough to warrant final conclusion, but the observations so far obtained are of value in two respects: They demonstrate that the basic formulas involved in the original studies and the method of attacking the problem have been sound. Of additional interest are the observations which have been made with great care to determine values of the roughness factor, n. Even with the data already at hand, additional values for this factor are always desirable, particularly for improved channels, as in this case.

It is to be noted that observed values of n for the improved channels ranged from 0.015 to 0.0285, but most of them were of the higher values. The low value of 0.015 has been readily explained. The originally adopted values were from 0.0225 to 0.025. The higher figures obtained from observation more nearly approach the value of 0.030 which has often been used as a general criterion for natural river channels of large sizes. Studies recently made along the Ohio River show values for this coefficient ranging from 0.035 to 0.047 in the main channel. These are, of course, in an unimproved reach of the Ohio.

The retarding basins, which formed the major parts of the Miami project, have demonstrated their effectiveness. While they are a departure from the generally accepted idea of storage basins, they have shown their value in cutting down peak flows, in distributing the flow over a longer period, and in demonstrating the fact that their operation is entirely automatic. The design of the conduits through the dams has been found to be sound and here, again, some interesting observations upon the value of n have been obtained, which check very closely those used in the design.

The engineers of the District are to be commended for thoroughness in obtaining rainfall and run-off data, and for the provisions being made for securing more of this information by additional stations. It is wisely pointed out that it is impossible to determine run-off from rainfall by any fixed rule or formula. The careful observations of all the factors entering into the determination of run-off will be of great value to those who do not have the facilities, either in time or money, to obtain the detailed field data for similar studies in other localities.

One of the most interesting features of the entire paper is that part which compares the actual run-off at Dayton, which occurred in March, 1927, with the probable run-off without the improvements. A reduction of 27% in the peak flow, which might have taken place, represents considerable less possible flood damage. The retarding basins and channel improvements along the Miami River have already proved beneficial to the District and from the observed data already obtained, it is apparent that the 1913 disaster can never be repeated.

WALTER M. SMITH,† M. AM. SOC. C. E. (by letter).‡—The writer has read this paper with a great deal of interest, because the dams, conduits, and spill-

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1371.

† Chf. Designing Engr., Div. of Waterways, Dept. of Purchases and Constr., State of Illinois; Cons. Engr., Greater Chicago Lake Water Co., Chicago, Ill.

‡ Received by the Secretary, June 15, 1928.

ways on that work were all designed under his direction. It is very gratifying to know that the six years of operation have justified the design and construction of the project as a whole as well as the individual units.

The writer has always had the idea that the value, $n = 0.013$, for the conduits would prove too high and that, therefore, more water would be discharged and less held in storage than the computed quantity. The concrete of the conduits was made very rich in cement—about 2 bbl. per cu. yd. of concrete—in order to secure a very dense and hard concrete that would resist abrasion and a growth of any kind of moss or other material. The material carried through the conduits during floods will naturally scour the floor somewhat, but should not affect the other parts of the surface. Therefore, the roughness factor, n, should not change very much over a long period. It is customary, in water supply or sewer computations, to assign to n a value a little higher than is ever expected in order to provide a factor of safety. In the case of the Miami Conservancy work, however, assigning too large a value to n does not give a factor of safety, but just the reverse. For example, the actual flow through the Englewood Conduit, as given in the paper, is $14\frac{1}{2}\%$ greater than as computed. Should this same ratio hold good at the Taylorsville and Huffman Dams, the total flow for the maximum flood through Dayton would be about 114 000 sec-ft. plus the accumulated flow below the dams, instead of about 100 000 sec-ft. plus this flow. As the channels through the towns were designed and built to accommodate the computed flow, this additional flow might cause damage.

It is regrettable that the author does not give results of observations at the Taylorsville and Huffman sites if they have been made, because the flow from these two sites combined is $7\frac{1}{2}$ times as great as that at Englewood. If none has been made, it would seem that in view of results at Englewood, observations at these two sites should be taken at as early a date as possible.

With regard to entry loss of head the conduits in all cases were built with a carefully designed bellmouth to prevent, not only loss of head at entrance, but to prevent vibration and booming caused by entrapping air just inside the entrance as would happen if the entrance had been built with a sharp edge. The vertical section through the Germantown Conduit (see Fig. 6*) shows this entrance with a very slight rounding, much less than it really has. As the author has not mentioned any trouble from vibration or booming at the entrances it is assumed by the writer that the bellmouth entrance is properly designed. This was one of the most serious features of the design and one of those most carefully studied.

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* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1377.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLARD BEAHAN, M. Am. Soc. C. E.*

DIED FEBRUARY 5, 1928.

Willard Beahan was born in Watkins, N. Y., on January 15, 1854, the son of James and Harriet (Griswold) Beahan. His grandfather was from the north of Ireland and of Scotch-Irish ancestry; his mother was of English descent.

Mr. Beahan prepared for college at Starkey Seminary, Starkey, N. Y., and was graduated in 1873. He entered Cornell University, Ithaca, N. Y., in 1874, from which he was graduated in 1878 with the degree of Bachelor in Civil Engineering. He took a prominent part in class and college activities, playing on his class football team and in his Senior year being elected Class Historian and Chief Engineer. After graduation he was elected to Sigma XI and Tau Beta Pi, honorary fraternities. He also became Life Secretary of his Class, and at the time of his death was preparing for the 50-year re-union.

After his graduation from Cornell University in 1878 he entered the service of the United States Corps of Engineers as Assistant Instrumentman and, later, as Computer on the Mississippi River Survey. In 1880, he was employed by the Gould Lines in the Southwest as a Rodman on the Texas and Pacific Railway. He was advanced to the position of Division Engineer in 1881 and then became Division Engineer on the Fort Worth and Denver City Railway where he was in charge of location and construction.

After an interval of one year, from March, 1883, to March, 1884, during which time he was employed as a Transitman on the Lehigh Valley Railroad, Mr. Beahan returned to the Southwest as Division Engineer on the Missouri Pacific Railway, where he remained until February, 1889, engaged in location, construction, and maintenance. He then went to South America with the North and South American Construction Company as Superintendent of Construction and Bridge Engineer of the Chilean Railway. Before the end of the year a revolution interfered with the progress of this work and Mr. Beahan crossed the Andes on a mule, made his way to Buenos Aires, Argentina, and returned to the United States *via* Europe.

During 1891, he was Superintendent of Streets of St. Louis, Mo. In after years, Mr. Beahan was accustomed to refer to this engagement as the time when he was political "boss" of St. Louis.

From November, 1891, to December, 1896, he was with the Anderson and Barr Company in various capacities. As President of the Anderson and Barr

* Memoir prepared by George H. Tinker, M. Am. Soc. C. E.

Clay Company, he developed a large and important business in the manufacture of clay products, in Streator, Ill. In 1896 and 1897 he was Lecturer on Railroad Location at Cornell and Leland Stanford, Jr., Universities.

From August, 1897, to May, 1898, Mr. Beahan was Superintendent of Construction on the Cascade Tunnel on the Great Northern Railway. From September, 1898, to September, 1900, he was again with the Lehigh Valley Railroad Company as Engineer of Maintenance, following which, he served as Division Engineer on the Chicago and Northwestern Railway until March, 1905.

From 1905 to 1924, Mr. Beahan was First Assistant Engineer on the staff of the Chief Engineer of the New York Central Railroad Company at Cleveland, Ohio. This long engagement ended with his retirement under the age and pension rules. Immediately thereafter, he entered the service of the "Nickel Plate Road" (New York, Chicago, and St. Louis Railroad Company) as Special Engineer in charge of relocation surveys. After apparently recovering from the effects of a serious operation, he continued to do consulting work for the Erie Railroad Company, but in the latter part of 1927 his health began to fail. He died on February 5, 1928.

In 1892, he was married to Bessie B. DeWitt, who survives him. He is also survived by two brothers.

Mr. Beahan's interest in Cornell University continued throughout his life. He was a member of the Cornell Society of Civil Engineers of New York and was one of the leaders in organizing the Northeastern Ohio Cornell Association. He was elected Alumni Trustee of Cornell University in 1900, and, again, in 1909.

In addition to Cornell and Leland Stanford, Jr., Universities, he lectured before many educational and technical societies. His interest in education and the guidance of young engineers never lessened. He was a member of the Board of Managers of the Cleveland Young Men's Christian Association and, for years, served as Chairman of the Committee on Technical Education of the Cleveland Engineering Society. His many talks to young engineers and students on the subject of Human Engineering were illustrated copiously by incidents from his own long and notable career.

During his long residence in Cleveland, Mr. Beahan became identified with the civic and professional life of the community. He was a member of the Congregational Church and was especially a leader and adviser of young men. He was a member of the Cleveland Engineering Society (President, 1908), Board of Civic League of Cleveland, Cleveland Chamber of Commerce, a Director of the Bureau of Municipal Research, and a member of the American Railway Engineering Association. He was also the first President of the Cleveland Section of the Society.

He was the author of "The Field Practice of Railway Location", and was also a contributor to the publications of the Society.

Mr. Beahan was elected a Member of the American Society of Civil Engineers on April 3, 1889, and served as a Director from 1919 to 1921.

CLARENCE AUSTIN CRANE, M. Am. Soc. C. E.*

Died December 5, 1927.

Clarence Austin Crane was born in New York, N. Y., on October 18, 1874. He was the son of Warren Cady and Caroline (Cleveland) Crane, and was descended from a long line of English ancestors, both great-great-grandfathers having served in the War of the Revolution.

After his preliminary education had been completed in the preparatory schools of New York, Mr. Crane attended the School of Mines, Columbia University, during 1891 and 1892. From October, 1893, to April, 1894, he was a student at Lehigh University, South Bethlehem, Pa., taking special courses in Mechanics.

Previous to this time, Mr. Crane had been connected with the Engineering Department of the New York, Susquehanna, and Western Railroad Company, but after leaving college he resigned this position to become Assistant Engineer with the Department of Parks of the City of New York. In this position, he was engaged in many important plans and improvements which included the Harlem River Driveway, the Zoological Gardens of Bronx Park, and the Spuyten Duyvil Parkway.

From 1900 to 1903, he served as Assistant Engineer with the New York Aqueduct Commission, on the construction of Jerome Park Reservoir. In 1903, he resigned from the city service and entered private practice as a Consulting Engineer. As such, he was engaged in surveying in the Boroughs of Manhattan and The Bronx, and elsewhere, and had charge of numerous grading, paving, and sewer contracts, etc. He also served as a City Surveyor in the Borough of Manhattan and was engaged in street grading, etc., on the upper part of New York City.

In 1909, Mr. Crane founded and organized the General Contractors Association, which through his efforts developed into the present influential organization. As Secretary of this Association, he was active in encouraging the enactment of statutes infinitely beneficial to contracting interests as well as to the State in general.

Although unfailing in his devotion to the Association, Mr. Crane's activities were by no means limited to this one undertaking. He was a student of and possessed an intimate knowledge of public affairs. He was a civil engineer with practical experience in engineering problems of design and construction. His advice and counsel were sought by his friends generally, as well as by public officials and members of the Association, to all of whom he was equally helpful in solving difficult civic, economic, and engineering problems.

He had a magnetism and charm of personality that won for him a loyal affection and respect, and made people seek to be with him. He was a devoted husband and father, constantly interested and eager for the welfare and happiness of his family. He is survived by his widow, Edna Montgomery

* Memoir prepared from information on file at Society Headquarters.

Crane, two daughters, Caroline Cleveland and Edna, and one son, Warren Cady, 2d; as well as his father, Mr. Warren Cady Crane, two sisters, Miss Sally B. Crane and Mrs. J. Harvey Birch, and one brother, Mr. Frank W. Crane.

Mr. Crane was a member of Mayor Walker's City Plan Committee, the Horatio Seymour Democratic Club, and the Sheriff's Jury, Third Panel. He was also a member of the Engineers Club, the Sons of the Revolution, St. John's Lodge No. 1, A. F. and A. M., and the Chapel of the Intercession of Trinity Church.

Mr. Crane was elected an Associate Member of the American Society of Civil Engineers on March 5, 1902, and a Member on May 4, 1909. He was also a member of the New York Section.

JAMES HENRY KENNEDY, M. Am. Soc. C. E.*

Died October 21, 1927.

The passing of James Henry Kennedy laid to rest one more of those fine old personalities who link this somewhat materialistic age with the romance of the railway builders of 40 or 50 years ago. That splendid group of pioneers have given much color to the history of National development on this Continent. Their part in those great enterprises which traversed the vast and beautiful expanse of Nature in the Northern and Western wilds, fashioned in them a nobility of character and purpose which has endeared them to all and rightly added to the pride and pleasure in their achievements. As Mr. Kennedy was an honored and respected representative of that passing generation of nation builders, it seems fitting that this brief narrative of his life should endeavor to retain at least some trace of the spirit which lifted him out of the humdrum of uneventful existence into adventure and success as a pioneer engineer in a realm of great and noble enterprise.

James Henry Kennedy was born on March 3, 1852, at Stittsville, Ont., Canada, which is just a few miles from Ottawa. His boyhood was spent on a farm in Elgin County, Ontario, near St. Thomas. There he developed a sturdy physique, a clean healthy mind, and a stirring ambition. His religious ideals were fostered in a Baptist atmosphere, and the habits of youth remained constantly with him to the end.

His early education was received at a rural school, and it was not until his twenty-fifth year that he commenced his higher education at Woodstock College, Ontario. Two years later, in October, 1879, he entered, as a Freshman, the old School of Practical Science, now recognized as the Faculty of Applied Science and Engineering of the University of Toronto. Under the personal instruction of the late Dean John Galbraith, with whom he formed a lasting friendship, he was graduated with honors in Civil Engineering in 1882, at the age of thirty. This marked the commencement of his professional career.

* Memoir prepared by P. H. Buchan, Esq., Vancouver, B. C., Canada.

It was typical of Mr. Kennedy's modesty that he habitually termed himself "an awful duffer at school", but the fact that he attempted and gained a university education without a single failure, at such a mature age, is significant of his strength of character and mental ability. During his latter days he was the recipient of much honor and frequent congratulations in recognition of his being the oldest living graduate of the School of Practical Science, and the possessor of the degree of Civil Engineer awarded to him by the University of Toronto in 1886.

His first engineering job was as Rodman and Field Draftsman on a survey for the Canadian Pacific Railway during the summer of 1882, but in September of that year he was appointed Assistant Engineer on the re-measurement of Section "B", and in April, 1883, was promoted to be Assistant Engineer on the location and construction of the Lake Superior Division, where he continued until March, 1885.

After spending three years in the wilds, Mr. Kennedy evidently felt the attraction of civilization for, in July, 1885, he was articled to A. D. Baikie, Provincial Land Surveyor, at St. Thomas, with whom he remained until he obtained his Ontario Land Surveyor's certificate (O. L. S.) in the spring of 1887. In May, 1887, he was re-engaged by the Canadian Pacific Railway Company as Locating Engineer on the Detroit Extension, between Woodstock and London, Ont., and afterward on the location and construction of the Temiscouata Railway in Quebec and New Brunswick. In May, 1889, he went to Minnesota to enter private practice in Winona as Deputy County Surveyor with the late M. S. Parker, M. Am. Soc. C. E.; but shortly afterward he was employed as Inspector of Tracklaying on the Neighart Branch of the Montana Central Railway.

For the next five years, Mr. Kennedy was very active on railway construction in the United States. In July, 1889, his services were engaged by the Great Northern Railway Company as Engineer in charge of surveys and construction on the Rocky Mountain Division of the Pacific Extension in Montana, through what is now known as Glacier National Park. To have progressed thus far, from an inexperienced Rodman on a survey party, in only seven years, points strongly to his enthusiasm and skill. He remained with this great project until March, 1892, after which he spent eighteen months on the construction of the Minneapolis, St. Paul, and Sault Ste. Marie Railway extensions in North Dakota. During this period he made examinations and reports on all the bridges in the "Soo" System for the late W. W. Rich, M. Am. Soc. C. E., then Chief Engineer of the "Soo" Lines.

In August, 1894, the spell was broken for the second time, and Mr. Kennedy resumed private practice as a Land Surveyor under his own name at St. Thomas. Concurrently, he became interested in a machine shop and bicycle factory in that city. However, three years of life in civilized parts appeared to satisfy him, for in February, 1898, he engaged with MacKenzie and Mann, under the direction of Mr. T. H. White, to go to the Stickine River in charge of Yukon Railway surveys between Telegraph Creek and Teslin Lake, in British Columbia. He remained there until mid-summer and then went south to take charge of the location of a line in the Okanagan,

from Penticton to Midway, B. C. Later, he made several surveys for the British American Coal Company in the vicinity of Crows' Nest Pass, which occupied him until the spring of 1900.

He spent two years in British Columbia, his future home, and then returned to Ontario, where he was engaged to locate an electric railway from London to Port Stanley, near the scenes of his boyhood. The call of the wilderness, however, soon lured him away to Michipicoton Harbor, where he took charge of three survey parties for the Algoma Central Railway Company, from the great Algoma iron mines to Sault Ste. Marie. It was during the winter of 1900-01 that he made a reconnaissance on snowshoes down the Agawa River to Agawa Bay, in company with the late William McCarthy.

Having reached the age of forty-nine years, Mr. Kennedy once more turned his face westward, this time to stay. In February, 1901, he re-entered the service of MacKenzie and Mann as Assistant Chief Engineer on the construction of the Vancouver, Victoria and Eastern Railway and Navigation Company's line from Penticton to Midway, on the location surveyed by himself in 1899. This line was acquired by the Great Northern Railway Company during 1901, and Mr. Kennedy acted in the same capacity with the latter Company. Nine years had elapsed since his previous engagement with the Great Northern in Glacier National Park. It was not long before he was promoted to the post of Chief Engineer of the Vancouver, Victoria, and Eastern Railway Company, from Laurier to the Pacific Coast, which position he retained until his retirement on August 15, 1915, at the age of sixty-three.

Much of his work in this connection involved heavy construction through rough mountainous country, the most difficult portions being the branch line from Laurier to Grand Forks, B. C., through the Kettle Valley, and the longer section of main line between Chopaka and Brookmere through the Similkameen and Tulameen Valleys. This construction called for a high degree of engineering initiative and skill to bring it to a successful conclusion, and Mr. Kennedy believed it to be the most difficult piece of railway construction he had ever undertaken. His retirement completed seventeen years with the Great Northern Railway Company on railway extensions.

In 1916, he opened an office for consulting practice at Vancouver, specializing in railway construction. He was also engaged on inspection work for the British Columbian Government during 1916 to 1917, on the construction of the Pacific Great Eastern Railway. From 1918 to 1922 he was retained by the Brooks, Scanlon, O'Brien Logging Company as Consulting Engineer on logging railway construction. In the summer of 1923 he made an inspection and report on the Edmonton, Dunvegan, and British Columbia Railway for the Alberta Government, and did further work of the same nature on this line in the late fall of 1926. During 1924 and 1925 he acted as Inspector of State Highway work for the Great Northern Railway Company at Rock Island, Wash.

That Mr. Kennedy kept in harness until he was nearly seventy-five years of age is no slight tribute to his splendid physical and mental qualities. His professional career covered a period of forty-five years. On January 20, 1887,

he joined the Canadian Society of Civil Engineers as an Associate Member, his election to Membership occurring on May 18, 1893. Toward the close of his career he was granted Life Membership by the Engineering Institute of Canada (the successor of the Canadian Society of Civil Engineers) and also by the Society.

On October 6, 1923, Mr. Kennedy sat for a group photograph comprising T. H. White and the late H. J. Cambie, Members of the Engineering Institute of Canada, and himself, in commemoration of their distinguished services as Pioneer Railway Builders in British Columbia. An enlargement of this group was presented by the Institute to the Provincial Library at Victoria in 1924. This group is unique for several reasons, not the least interesting being the fact that Mr. Kennedy, although seventy-two, was the youngest member of the trio.

Among his varied interests during his later years, he was Vice-President of the Pacific Investment Corporation of Vancouver, and served as a member of the Executive of the Convocation of the University of British Columbia. He was a member and regular attendant of the First Baptist Church of Vancouver.

On June 30, 1884, he was married to Annie Huntley, at Toronto, Ont. He is survived by his widow, two sons, Edward and Arthur, and one daughter, Alice (Mrs. Harold P. Cowan, of Toronto). There are four grandchildren. Mr. Kennedy failed noticeably in his last year, although he believed that a contemplated trip to California would restore his former vigor. He passed away quite suddenly at his home in Vancouver and was buried in Ocean View Cemetery, leaving to his native Canada a long record of honorable achievement in his chosen career of Pioneer Railway Builder.

Mr. Kennedy was elected a Member of the American Society of Civil Engineers on May 2, 1900.

FRANK OSCAR MAXSON, M. Am. Soc. C. E.*

Died January 21, 1928.

Frank Oscar Maxson was born in Stillmanville, Conn., on August 8, 1851, of pioneer stock. When he was three years of age his family, influenced by the Western movement of the times, went to California, making the journey by the long circuitous route of the Isthmus of Panama. Eight years were spent in the mining districts of California and Mr. Maxson's two younger brothers and a sister were born there. In 1862, however, ill health compelled his mother to return to her home in Rhode Island, and the four children accompanied her.

After his mother's death, Mr. Maxson and his brother, Louis, "boarded out", helping to support themselves by carrying papers before school in the mornings, the younger children being cared for by relatives. The two older boys attended the public schools of Norwich, Conn., and, later, the Norwich Free Academy; Mr. Maxson then entered the Sheffield Scientific School of

* Memoir prepared by DeWitt C. Webb, M. Am. Soc. C. E.

Yale University, at New Haven, Conn. Later he returned to the Norwich Academy and taught for one year, after which he re-entered Yale for a year of post-graduate work.

After he had completed his studies, Mr. Maxson began his long and faithful service to the United States Government, as Topographer of a party engaged in surveying the States of Colorado, New Mexico, and Arizona. The work also included a survey of the Yosemite Valley in California, and Mr. Maxson made one of the earliest authentic maps of that beautiful and far-famed region. He continued in this work for seven years, half of each year being spent "in the field", the other half in Washington, D. C., plotting and computing the data which he had acquired.

In October, 1881, Mr. Maxson successfully passed an examination for appointment to the Corps of Civil Engineers, U. S. Navy. He was commissioned on October 29 of that year and assigned to duty at the Navy Yard, Pensacola, Fla. He served successively at the New York, Boston, Mass., Washington, and Mare Island, Calif., Navy Yards. During the early days of the war with Spain he was sent to the Norfolk, Va., Navy Yard where he served during the war period. He attained the rank of Lieutenant Commander in 1898, Commander in 1901, and Captain in 1906. Following the close of the war with Spain, Captain Maxson returned to the Boston Yard.

In 1901, he was ordered to the Philippine Islands to take charge of public works and public utilities at the Naval Stations at Cavite and Olongapo. He returned to the United States early in 1904 and was detailed to the League Island Navy Yard at Philadelphia, Pa. Following subsequent duties at Portsmouth, N. H., and Key West, Fla., he was transferred to the retired list with the rank of Captain in August, 1913. In 1915, he was recalled to active duty and detailed to assist the Department of Justice in Washington in the prosecution of several lawsuits involving Navy contracts. He completed this duty in February, 1917, and reverted to an inactive status on the retired list.

The outbreak of the World War made it necessary again to recall Captain Maxson to active service. In October, 1917, he was placed in charge of all Naval public works in the Seventh Naval District, with headquarters at Key West. He performed meritorious service in this capacity, having charge of the construction of many important Naval war-time projects, including quay walls, piers, barracks, hospital buildings, and the Air Station, at Miami, Fla. He was relieved from active duty in October, 1919, and shortly thereafter moved to Coupeville, Wash., where he resided at the time of his death. He contracted pneumonia about the middle of January, 1928, and passed away after a few days' illness.

Captain Maxson's career as a civil engineer was praiseworthy. All his work exemplified his careful foresight and professional attainments. His patriotism and generous consideration for his fellow men were outstanding characteristics. Any undertaking for the welfare of his country or community received his whole-hearted support.

On December 26, 1877, he was married at Washington, D. C., to Evelyn M. Van Doren. He is survived by his widow, two sons, Dr. Frank T. Maxson

and Dr. Louis H. Maxson, of Seattle, Wash., and one daughter, Evelyn Maxson, of Coupeville, Wash.

His home life was ideal and continued for over half a century. On December 26, 1927, the golden wedding anniversary of Captain and Mrs. Maxson was celebrated at their home in Coupeville, at which all the members of the immediate family were present.

Captain Maxson's natural gifts combined with the wide range of his experience and reading made him an interesting conversationalist on a great variety of subjects. He had a most agreeable personality and a keen understanding of human nature. These traits, together with a delightful sense of humor which was habitually expressed in good-natured banter, deeply endeared him to all.

For many years Captain Maxson was an Elder of the Metropolitan Presbyterian Church, of Washington, D. C., and at the time of his death was a Trustee in the Coupeville Congregational Church. He also taught large Sunday School classes in Washington, Key West, and elsewhere. He was not a man whose religious spirit was confined to the church, but rather one who carried it with him into his everyday life, as shown by his innumerable kind deeds and generous service for others.

Captain Maxson was elected a Member of the American Society of Civil Engineers on May 1, 1889.

MORRIS JOHN RIGGS, M. Am. Soc. C. E.*

DIED FEBRUARY 7, 1926.

Morris John Riggs was born on his father's farm at Horton, Bremer County, Iowa, on January 14, 1862. His parents were William and Sophronia Riggs, early pioneers of that State. He was educated in the district schools of Bremer County, in the High School, at Waverly, Iowa, and in Iowa State College, at Ames, from which he was graduated in June, 1883, with the degree of Bachelor of Civil Engineering, having specialized in bridge engineering. In 1905, he obtained from his Alma Mater the degree of Civil Engineer.

Before entering college in 1880, Mr. Riggs had spent four years teaching in the district schools. He worked his way through college, milking cows and performing other services on the Agricultural Farm. He was also the first Business Manager of Athletics at Ames, and was known as the "Father of the Memorial Union", of which he was President for five years, later becoming Honorary President. It was largely due to his tireless efforts that more than $1 000 000 was subscribed to the Union. As an Alumnus he always retained great interest in his Alma Mater, making frequent visits to the college. From 1920 to 1924, he served as President of the Alumni Association.

* Memoir prepared by Walter J. Sherman and George A. Taylor, Members, Am. Soc. C. E.

Following his graduation Mr. Riggs was employed for about a year making county maps in Kansas. In the spring of 1885, he became Bridge Engineer for Mr. S. M. Hewett, Agent, at Minneapolis, Minn., for the Smith Bridge Company of Toledo, Ohio. In July, 1887, Mr. Riggs was appointed Draftsman, Designer, and Estimator for this Company and continued in that position until April, 1893, when he became Chief Engineer of the Toledo Bridge Company, the successor to the Smith Bridge Company. The Toledo Bridge Company, in turn, was succeeded by the American Bridge Company and in April, 1901, Mr. Riggs was appointed Manager of the Toledo Plant of this firm, in which capacity he served until his death. In connection with his duties he had charge of numerous important works, among which was the design and construction of the Toledo Plant of the American Bridge Company. During the many years he served these several bridge companies, he also acquired splendid experience in bridge engineering.

At the time of his death Mr. Riggs was President of the Toledo Section of the Society, of the Chamber of Commerce, of the Young Men's Christian Association, at Toledo, and also of the Ohio State Young Men's Christian Association. He had served the local organization as President for sixteen years and was very much interested in its work, serving also as a member of its National Council. He was a member of the Ashland Avenue Baptist Church and leader of its Men's Class. During the World War he was very active in the several Liberty Loan Campaigns. On one occasion (in 1910), without solicitation he was nominated for Mayor of Toledo on the Progressive Ticket. Mr. Riggs was also a member of the Inverness Club of Toledo.

On October 11, 1893, Mr. Riggs was married to Alma M. Fassett, daughter of the late Elias Fassett, an early pioneer of Toledo.

On Thursday, February 4, 1926, Mr. Riggs complained of an earache while returning from a trip to Detroit, Mich. He became unconscious on Friday and remained so until his death on Sunday, February 7, 1926, the cause of which was announced as spinal meningitis. He is survived by his widow, Alma Fassett Riggs; a sister, Sarah M. Riggs, Head of the Department of History in the State Normal School, at Cedar Falls, Iowa; another sister, Mrs. Luderne Franklin, of Chicago, Ill.; and one brother, Marvin L. Riggs, who lives on the old 240-acre homestead at Horton, Iowa.

On February 9, 1926, memorial services were held at the Ashland Avenue Baptist Church, attended by delegations representing the Young's Men's Christian Association, the Iowa State College, the Chamber of Commerce, the Rotary Club, and the American Bridge Company. Testimonials were given by officers of the Church and Sunday School and by members of the numerous delegations present.

At Rotary, silent prayers were offered and an appropriate tribute was delivered. At the time of Mr. Riggs' funeral the Chamber of Commerce was closed out of respect to the memory of one of its most valuable members, and the City Council declared in a resolution that he had been one of the City's outstanding civic leaders. His was a "life widely useful, strangely courageous, and always busy with a tenderness, persistency, and loyalty known to few men".

In the death of Morris John Riggs, the City of Toledo has lost an ideal citizen, the American Bridge Company, an efficient Managing Engineer, the Chamber of Commerce, an earnest worker for the interests of his adopted City, the engineers of Toledo a sympathetic friend, the Young Men's Christian Association of Toledo, a most liberal supporter, and the Church to which he belonged, an earnest Christian laborer.

Mr. Riggs was elected a Member of the American Society of Civil Engineers on December 26, 1899.

GLENN MASON SCOFIELD, M. Am. Soc. C. E.*

DIED DECEMBER 21, 1926.

Glenn Mason Scofield was born in Hermon, N. Y., on May 3, 1873, the son of George Vincent Scofield and Emily Jane (Mason) Scofield, both of whom were of typical New England stock, their families having landed in Connecticut and worked their way northwestward to Northern New York. His father, who was born in Jefferson County, New York, on October 20, 1831, was a Hydraulic Engineer at a time when small power plants were the rule and hydraulic engineering was in its infancy. His mother was a woman of beautiful character, a school teacher before her marriage, and always a faithful worker in the church and social affairs of her town.

Mr. Scofield's early education was acquired in the Grammer School and High School at Hermon. He then entered the Engineering Department of Union College, at Schenectady, N. Y., from which he was graduated in 1897 with the degree of Civil Engineer.

His first professional experience was as Engineer with the Youngstown Bridge Company, after which he became Assistant Manager of the New York Office of the Company, and, finally, Manager of this office, handling work throughout the Eastern States.

In 1903, he and his brother, Edson Mason Scofield, M. Am. Soc. C. E., formed the Scofield Engineering Company of Philadelphia, Pa. A few years later the two brothers associated with themselves, Mr. Frank Daugherty, a Nationally known electrical engineer. In 1920, these three, with Mr. Ford J. Twaits, a well-known structural engineer, organized the Scofield Engineering Construction Company, of Los Angeles, Calif. These principals have handled engineering and construction work amounting to more than \$300 000 000, of which the following are samples: A coaling station for the U. S. Navy, at Guantanamo, Cuba; the steel work for a coaling station for the U. S. Navy, at Manila Harbor, Philippine Islands; dry docks for the U. S. Navy, at League Island Navy Yard, Philadelphia, Pa., and at the Mare Island Navy Yard, San Francisco, Calif.; power stations for the Harwood Electric Company, at Hazleton, Pa., and for the City of Jacksonville, Fla.; concrete ships for the U. S. Shipping Board, during the World War, at San Diego, Calif.; the Pacific Mutual Life Insurance Building; the Biltmore Hotel; the Biltmore Theatre; the Hellman Bank Building; the Pershing Square Building; the

* Memoir prepared by J. C. Davenport, Cost. Engr., Scofield Eng. Constr. Co., Los Angeles, Calif.

Elks Club Building; the Barker Building; the Commercial Club; the Ebell Club; the Hospital of the Good Samaritan; the Title Insurance and Trust Building; and the Sears-Roebuck Buildings, all of Los Angeles, Calif.; the Cooper Arms Apartments, at Long Beach, Calif.; and the Pacific Southwest Trust and Savings Bank, at Pasadena, Calif., and many other similar examples of high-class construction.

Mr. Scofield had a wonderful personality and a great sense of humor, he was a good mixer and the life of the party wherever he might be. Although he was educated as an engineer, he early had concentrated his efforts on the selling end of the business in which his temperament and character made him specially effective. He never forgot a face and took such a personal interest in every new acquaintance that he could repeat the details of the meeting after years of separation.

Had he been spared longer, undoubtedly his works would have been greatly enlarged, although now they stand as an impressive monument to his talent and industry, and to that of his associates. Mr. Scofield is survived by his partners, Messrs. Edson Mason Scofield, Ford J. Twaits, and Frank Daugherty, who are carrying on the work he so ably helped to create. He was a member of the Union League of Philadelphia, and the University Club, the Elks Club, the Commercial Club, the Jonathan Club, and the Breakfast Club of Los Angeles.

He was married at Youngstown, Ohio, on September 14, 1899, to Rosa May Clevely. He died at Los Angeles on December 21, 1926, while in the prime of life. He left, in addition to his widow, a daughter, Marion Adele, and a son, Glenn Mason Scofield, Jr.

Mr. Scofield was elected a Member of the American Society of Civil Engineers on September 7, 1904.

KIRBY LEE STRICKLAND, M. Am. Soc. C. E.*

Died December 10, 1927.

Kirby Lee Strickland, the son of Thomas and Frances (Sewell) Strickland, was born on November 12, 1871, in Coweta County, Georgia. He was educated at the North Georgia Agricultural College, a branch of the University of Georgia, where he was in attendance three and one-half years.

In July, 1896, Mr. Strickland entered the employ of the Edgemoor Bridge Works, as Timekeeper and Assistant Engineer on the erection of steel structures, with which Company he remained until 1901, at which time the American Bridge Company was organized. The Edgemoor Bridge Works was one of the various companies which were amalgamated into the American Bridge Company, and on account of his field experience Mr. Strickland was appointed to inaugurate an accounting system for the Erecting Department of the newly organized Company. He was subsequently transferred to field duties, where he remained until late in 1901, when he entered the General Office at Pencoyd, Pa., as Assistant to Manager of the Erecting Department.

* Memoir prepared by the following Committee of the Illinois Section: Messrs. Albert F. Reichmann, A. F. Robinson, and O. F. Dalstrom, Members, Am. Soc. C. E.

In April, 1904, Mr. Strickland was appointed Assistant to the Manager of the Eastern Division Erecting Department, with offices in Philadelphia, Pa., in which capacity he was employed until July, 1915. During this time he supervised the erection of steel structures of every description, including the Baltimore and Ohio and the Pennsylvania Railroad Bridges over the Susquehanna River at Havre de Grace, Md., and numerous bridges on the New York, New Haven and Hartford, the Philadelphia and Reading, and the Southern, Railroads.

In July, 1915, he was appointed Division Erecting Manager of the American Bridge Company, at Chicago, Ill., which position he held until the time of his death. During this period he was in charge of the erection of many large and important structures, a few of which are the Chicago, Burlington and Quincy Railway Bridge across the Missouri River, at Kansas City, Mo.; the Union Pacific Bridge across the Missouri River, at Omaha, Nebr.; the St. Joseph and Grand Island Bridge across the Missouri River, at St. Joseph, Mo.; the Bismarck and Mandan Highway Bridge, over the Missouri River at Bismarck, N. Dak.; the Atchison, Topeka and Santa Fe Railway Bridge across the Mississippi River, at Fort Madison, Iowa; the large steel ore docks at Duluth and Two Harbors, Minn.; the Illinois Steel Company's open hearth buildings, at Gary, Ind., and South Chicago, Ill.; and many other erection projects for the United States Steel Corporation. During the World War Mr. Strickland was active in the erection of various projects for supplying the United States Government with war materials, notably the large powder plant for the du Pont de Nemours Company, at Nashville, Tenn.

The last, and one of the most important, structures erected under Mr. Strickland's supervision was the large highway bridge across the Carquinez Straits of San Francisco Bay, which was completed during the summer of 1927. It was while he was on a trip in connection with the completion of this work that he was taken sick with what proved to be his final illness.

Mr. Strickland was a man of pleasing personality with numerous friends and admirers, who knew him also as a man of unusual resourcefulness resulting from his broad experience. He was an interesting narrator of experiences and anecdotes, and took keen delight in relating stories pertaining to his boyhood days in the "Old South." He was held in high esteem by all his associates throughout his long term of service with the American Bridge Company, and he numbered among his friends many men prominent in the Engineering Profession throughout the United States. He was a member of the Western Society of Engineers and the Engineers' Club of Chicago.

Mr. Strickland had made his home in Evanston, Ill., a suburb of Chicago, since 1915. On March 16, 1907, he was married to Charlotte Barney, of Pottstown, Pa., who died in 1921. Two daughters by this marriage, Eleanor and Elizabeth, survive him. On November 17, 1923, he was married to Amy Louise Halvorsen, of Chicago, who, with a son, Kirby Lee, survives him. Mr. Strickland is also survived by his mother, Mrs. Ellen Byram, who has remained at the ancestral home in Georgia.

Mr. Strickland was elected a Member of the American Society of Civil Engineers on October 15, 1923.

EDWIN HALL WARNER, M. Am. Soc. C. E.*

Died June 17, 1927.

Edwin Hall Warner was born in New York, N. Y., on February 21, 1858, the son of William H. and Anna Pamela (Conger) Warner. He attended the University of the City of New York, (now New York University), and was registered in the Polytechnic Engineering Department of the School of Applied Science.

Mr. Warner's technical career was varied and important. During the period from 1884 to 1886, he was engaged on the Mexican National and Mexican Central Railways, as Instrumentman and Assistant Engineer in charge of various construction works. During 1887 and 1888 he was Assistant Engineer and Acting Division Engineer, Maintenance of Way, for the Union Pacific Railway Company, with headquarters in Denver, Colo. In 1888 and 1890 he acted as Assistant Chief Engineer for the Seattle, Lake Shore and Eastern Railway Company (now the Northern Pacific System), with headquarters in Seattle, Wash. During this engagement he located and built 110 miles of railway from Snohomish, Wash., north to the Canadian boundary.

From 1891 to 1898 Mr. Warner was in private practice in Seattle, engaged in the development of mining properties, hydraulic power investigations, and installation of mining power plants; and for a part of the time, he acted as City Engineer for Seattle, on extensive construction of sewers. From May, 1898, to January, 1899, he was Assistant Chief Engineer for the White Pass and Yukon Railway Company in charge of location north of the Canadian Boundary. He located and built the first miles of track extending out of Skagway. Mr. C. E. Hawkins was Chief Engineer of this extremely difficult and arduous work.

From 1899 to 1903 Mr. Warner was engaged in private practice in Republic, Ore., as Engineer for the Republic Mining Company, and also on various mine investigations and installations. In 1903, he became Principal Assistant Engineer for the Columbia Improvement Company (Stone and Webster Syndicate), on the Puyallup Power development at Electron, Wash., the Engineer in charge being Mr. J. C. Cunningham who was succeeded by George S. Binckley, M. Am. Soc. C. E. This was a 20 000 h.p. installation, and during the early stages the forces under Mr. Warner's supervision included as many as 1 400 men.

In 1904 he moved to California and became Chief Engineer of the Abbot Kinney Company, at Venice, supervising the canals and bridges for this, one of the early play areas of Los Angeles. In this period Mr. Warner was associated with the late James Dix Schuyler, M. Am. Soc. C. E., and, in 1905, became Chief Engineer of the Tri-State Land Company, at Scottsbluff, Nebr. —a project for which Mr. Schuyler was Consulting Engineer. The location of 100 miles of canal, for a 1 750-sec. ft. flow, and the construction of 25 miles under his direct supervision, was part of Mr. Warner's accomplishment. From

* Memoir prepared by William S. Post, Assoc. M. Am. Soc. C. E., and Philip Schuyler, M. Am. Soc. C. E.

June, 1906, to January, 1907, he again undertook construction at Mr. Schuyler's instance. He became Assistant Chief Engineer of the Mexican Light and Power Company, for the construction of the dam at Necaxa, Mexico, where he had responsible charge of dam, tunnel, conduit, and machine shops and the construction of 22 miles of railway.

From 1907 to 1910 he was engaged in private practice in Los Angeles, Calif., designing irrigation projects, buildings, and ocean piers. From 1910 to 1916 he was Constructing Engineer for the Southern California Edison Company, with headquarters at Los Angeles. His duties included the construction of the 65 000 h. p. generating station at Long Beach, Calif., about twenty-five sub-stations, and the direct supervision of a large power tunnel on the Kern River. From 1916 to 1919 he was again in private practice, acting, among other engagements, for the American Trona Company at San Pedro and Searles Lake, Calif., in potash production.

From 1919 to 1921 Mr. Warner served as Construction Engineer for two important concrete dams in California, namely, the Kerckhoff Dam, of the San Joaquin Light and Power Company, on the San Joaquin River, and the Snow Mountain Power Company Dam on the Eel River.

In 1922, he re-entered private practice, first maintaining an office in San Francisco, and, finally, removing to Burlingame, Calif., where he and Mrs. Warner established a delightful home. It was in his office in Burlingame that he died of heart failure on June 17, 1927.

In Burlingame, Mr. Warner was also able to devote himself to civic affairs, for which his broad outlook, enjoyment of social and economic problems, fine tact, and personality, peculiarly fitted him. It is notable in the career of this engineer, absorbed in the main with the interests and perplexities of his clients, that he also gave generously to public service. An editorial, published a year before his death testifies to the thorough attention and organizing ability, which was an element of his success, and also to the appreciation of the community:

"A Word of Praise*"

"When a man has labored faithfully and well in public service, and particularly when that service has been unremunerative and has been performed in a quiet, unassuming sort of way, a word of praise is not amiss.

"Edwin H. Warner, last Monday night, tendered his resignation to the City Board of Trustees, after years of duty in the position of Secretary of the Park and Playground Commission. The plain truth of the matter is that Warner did practically all the work in connection with the functioning of the Commission and that he accomplished an amazing amount during the past two years.

"He resigned simply because the duties of the Commission had become an increasing burden, encroaching upon his private business, and because two years of free and voluntary labor at such a task certainly entitles a man to the privilege of relinquishing the responsibility to some one else.

* * * * * * * *

"Warner, on his own initiative, made practically a complete survey of all the trees in Burlingame. He consulted experts on the culture and diseases

* *Burlingame Advance-Star*, May 19, 1926.

of trees, determined the best trees for the city, and approached the entire problem in a thorough and scientific manner.

"And the main burden of the work has been the innumerable complaints and requests of property owners concerning their own tree problems. Warner maintained the consistent policy that these property owners were entitled to an immediate answer. This involved almost daily investigation on his part, the continual giving of time and energy to the work, and a large amount of correspondence. And that at no small sacrifice for a busy professional man to make.

"It is a matter of record that most men who have labored so faithfully in the service of the public come sooner or later to the disheartening realization that appreciation of their efforts is very seldom expressed by their fellow men and that the general public is relentlessly stingy with its praise. People walk miles or write page after page to kick about something, but they usually have no time or energy for commendation. Bearing this in mind, this is only an attempt to give a bit of honest praise where it is deserved."

Technically, Mr. Warner was known as a brilliant Location Engineer. In administrative work and construction his strongest forte perhaps was initiative. He possessed great organizing ability, in which he exercised a personal charm and sense of humor, which appealed not only to his friends, but to construction foremen and laborers as well. His tastes were fine; he was cultivated and very well read; and an excellent conversationalist.

One of the writers had the good fortune, as City Engineer of Ocean Park and Venice, from 1904 to 1905, to become intimately acquainted with Mr. Warner, later following him to Nebraska as an Assistant on the construction of the Tri-State Land Company Canal Project. Mr. Warner was a tall, slender, erect, handsome man, dignified and austere on first acquaintance, but possessed of a rare sense of humor and the ability to "play". He frequently organized minstrel shows, acting equally well the part of interlocutor and end-man. In short, he could play with his subordinates and at the same time retain their respect.

Mr. Warner was married on August 21, 1890, at Seattle, to Frances B. Ferguson, the daughter of Colonel David Ferguson, a distinguished figure in the Sixties in California and later of the American Colony in the City of Mexico. With sympathy and pride in her husband's engineering achievements Mrs. Warner was a constant companion in his various engagements in the Western United States and Mexico.

Mr. Warner was elected a Member of the American Society of Civil Engineers on October 4, 1893.

BERTRAND THORP WHEELER, M. Am. Soc. C. E.*

Died March 20, 1928.

Bertrand Thorp Wheeler was born in Lempster, N. H., on November 25, 1863, the son of Daniel Bingham and Maria Wheeler. His boyhood days were spent in Cambridge, Mass., where he attended the public schools until

* Memoir prepared by A. H. Morrill, M. Am. Soc. C. E.

he reached the age of 15. In 1879, he entered Dartmouth College at Hanover, N. H., where he took a scientific course, and was graduated in 1884 with the degree of Bachelor of Science.

Mr. Wheeler's first employment was in 1885 as Rodman with the Old Colony Railroad Company, now part of the New Haven System. In 1886, he was made First Assistant to the Chief Engineer of the "Old Colony", which position he held until 1895. During this period he had charge of second-track construction, grade separation, and other minor work. He originated and promoted the plans for separation of grades at West Fourth Street, Boston, Mass., involving a new bridge over Fort Point Channel which was among the first works of this character accomplished in Massachusetts.

From 1895 to 1896, and from 1900 to 1901, Mr. Wheeler held the office of Superintendent of Streets for the City of Boston under the Republican administrations of Mayors Curtis and Hart, making a splendid record in this onerous position. He was engaged in private practice from 1896 to 1900.

In 1902 he entered the service of the New York, New Haven, and Hartford Railroad Company as Assistant Engineer of Construction, which position he held until 1907, when he was made Engineer of Construction. During this period he had charge of the work in connection with the elimination of grade crossings at Dudley Street, and Savin Hill to Neponset (Boston), Hyde Park, Attleboro, Worcester, and other points.

From 1910 to 1912, Mr. Wheeler was also Engineer of Construction of the Boston and Maine Railroad Company. On November 1, 1912, he severed these connections and accepted the office of Chief Engineer of the Maine Central Railroad Company (succeeding the late Mr. T. L. Dunn), taking up his residence at Portland, Me. He was also Chief Engineer of the Portland Terminal Company, a subsidiary of the Maine Central. In this position he was responsible for the condition of right of way and structures on more than 1 100 miles of road, for property valued at nearly $50 000 000, and at times he had more than 3 500 men working under his direction.

During his service as Chief Engineer of the Maine Central Railroad, the line was extensively modernized and many new structures were created under his direction, some of the more notable of which were the double-tracking of the main line between Clinton and Waterville, Me., the erection of railway bridges across the Kennebec River at Augusta and Fairfield, and the construction of the Rigby Terminal Yards, in South Portland, Me.

Mr. Wheeler was a man of brilliant intellect, sound sense, and good judgment, possessing a rare faculty of analyzing a problem or looking over a plan and intuitively picking out any flaws it might contain. He was noted for the courage of his convictions and would stand by his guns under fire. His intolerance of incompetency, the bluntness and directness of his judgments, and his ever-constant desire for results often caused him to be misjudged by those who did not know him intimately. His associates and employees could see farther than the rather brusque exterior of the man, recognizing and prizing his uniform and constant justice, impartiality, and loyalty. Among them the feeling of sadness for a friend departed is tempered by the thought that his work and his memory will long endure.

Mr. Wheeler had not been in the best of health for two years previous to his death. On February 22, 1928, he was operated on for sinus and antrum trouble at the Maine Eye and Ear Infirmary, in Portland. Although he rallied from the operation, the respite was brief and he failed gradually, passing away on March 20.

In 1888, he was married to Mabel Cole, the daughter of George H. Cole, proprietor of the American House, at Fitchburg, Mass., by whom he is survived. He also left six children, Bertrand C. Wheeler, of Chicago, Ill., Allan T. Wheeler, of Wellesley Hills, Mass., Donald B. Wheeler, of Buffalo, N. Y., Mrs. Warren B. King, Mrs. Theodore K. Thurston, and Miss Ruth Wheeler, of Portland. Mr. Wheeler was always a devoted family man and took great interest in his summer home which was beautifully located on Great Diamond Island in Casco Bay, near Portland.

He was a member of the Boston Society of Civil Engineers, the Dartmouth Club of Boston, and the Massachusetts Society of Colonial Wars and Sons of the American Revolution.

Mr. Wheeler was elected a Member of the American Society of Civil Engineers on June 30, 1910.

FREDERICK BAIRD ADAMS, Assoc. M. Am.Soc. C. E.*

Died November 13, 1927.

Frederick Baird Adams was born on March 6, 1875, at Reading, Pa., the son of E. Ralph and Loretta (Loag) Adams. His education was acquired in the public schools of Reading.

In September, 1893, Mr. Adams entered the service of the Philadelphia and Reading Railway Company as Rodman, and in March, 1894, he accepted a position in the same capacity with Mr. W. H. Dechant, of Reading. From September, 1894, to June, 1895, he was a student of civil engineering at the University of Pennsylvania, following which he became Timekeeper with Mr. O. N. Wieand, on the construction of heavy masonry walls and foundations at Reading.

From January, 1896, to May, 1897, Mr. Adams served as a Clerk in the Maintenance-of-Way Department of the Philadelphia and Reading Railway Company. In 1897, he was engaged as a Transitman for the City of Reading in charge of sewer and other municipal work. Subsequently, he was appointed Engineer in charge of the construction of house sewers for the City of Reading, which position he held until April, 1900.

In July, 1900, Mr. Adams accepted a position as Assistant Supervisor for the Philadelphia and Reading Railway Company and, in December, he was appointed as Supervisor at Shamokin, Pa. In March, 1910, he was transferred to Pottsville, Pa., and on November 1, 1925, he was made Supervisor at Reading, which position he held at the time of his death.

* Memoir prepared by John S. Goodman, M. Am. Soc. C. E.

He was a remarkably fine type of man, fond of his home and family, and active in his affiliation with the Protestant Episcopal Church. He was very popular with his business associates. He was always attentive to his duties, and showed considerable energy and capacity for work. He was particularly successful in the construction of the trackage in the Saint Clair Yards of the Reading Company, in 1910, and for several years thereafter. His general characteristics were those of a student, and he specialized at one time in bridge design.

He died on November 13, 1926, at the Homeopathic Hospital, in Reading, after a brief illness.

Mr. Adams was married on October 6, 1903, to Lulu G. Felix, of Reading, who survives him.

He was a member of the American Railway Engineering Association, the Engineers' Society of Pennsylvania, and a Certified Member of the American Association of Engineers.

Mr. Adams was elected an Associate Member of the American Society of Civil Engineers on August 31, 1925.

MARIO JOHN MIDOLO, Jun. Am. Soc. C. E.*

DIED MARCH 11, 1926.

Mario John Midolo, the son of John and Kathryn Midolo, was born in Syracuse, Italy, on May 21, 1900, and received his primary education in his native country. When the boy was seven years old, the family came to America and settled in New York, N. Y., where he continued his elementary education in the public schools. This at first proved quite difficult as he was not well versed in the English language; but it was not long before he was one of the most promising students in the school he attended.

After remaining in New York about five years, the family removed to Philadelphia, Pa., where Mr. Midolo entered the Drexel (Public) School, and finished his elementary course in 1915. At this time, the question of higher education and his future profession was uppermost in his mind. He then entered the South Philadelphia Manual Training High School and finished the course there which brought him nearer his goal, namely, the profession of Civil Engineering. He was graduated well up in his class in February, 1919, and owing to the record that he had made while a student, he was chosen to enter Pennsylvania State College, in September, 1919.

This proved to be another stepping stone toward his ambition, and his untiring efforts and application were soon rewarded. He was graduated from Pennsylvania State College in June, 1923, with the degree of Bachelor of Science in Civil Engineering.

Engaging in his profession, Mr. Midolo first served as Assistant to the Superintendent on Sanitary Sewer Construction for the Perna Construction

* Memoir prepared by Francis A. Cotney, Esq., Philadelphia, Pa.

Company. Later, he followed Land Surveying and was associated with Mr. Thornton Hollingshead, of Morristown, N. J. Shortly afterward he accepted a position as Field Engineer with the Barber Asphalt Paving Company, of Philadelphia.

As he had now had considerable professional experience, he felt he would like to make a permanent connection. Accordingly, Mr. Midolo took the civil service examination in connection with the Bureau of Surveys, City of Philadelphia, and was successful in securing an appointment in one of the District Offices (7th Survey District), where he remained until September 15, 1925. He then accepted a position as Engineer and Estimator for a private contractor, Mr. Frank Brennan, who is well known in South Philadelphia.

It was while in Mr. Brennan's employ that Mr. Midolo contracted his fatal illness. Shortly before Christmas, 1925, he caught a heavy cold and was compelled to remain at home for a time, but his energy and love for his work took him back to the office. Although ambitious to continue his work, his health gradually began to decline, and he was compelled to resign his position.

He died on March 11, 1926, and is survived by his father and several brothers and sisters, all residents of Philadelphia. Modest and unassuming, and devoted to his family, he was cut off at the beginning of a promising career.

Mr. Midolo was elected a Junior of the American Society of Civil Engineers on December 15, 1924. He was also a member of the Penn State Club of Philadelphia.